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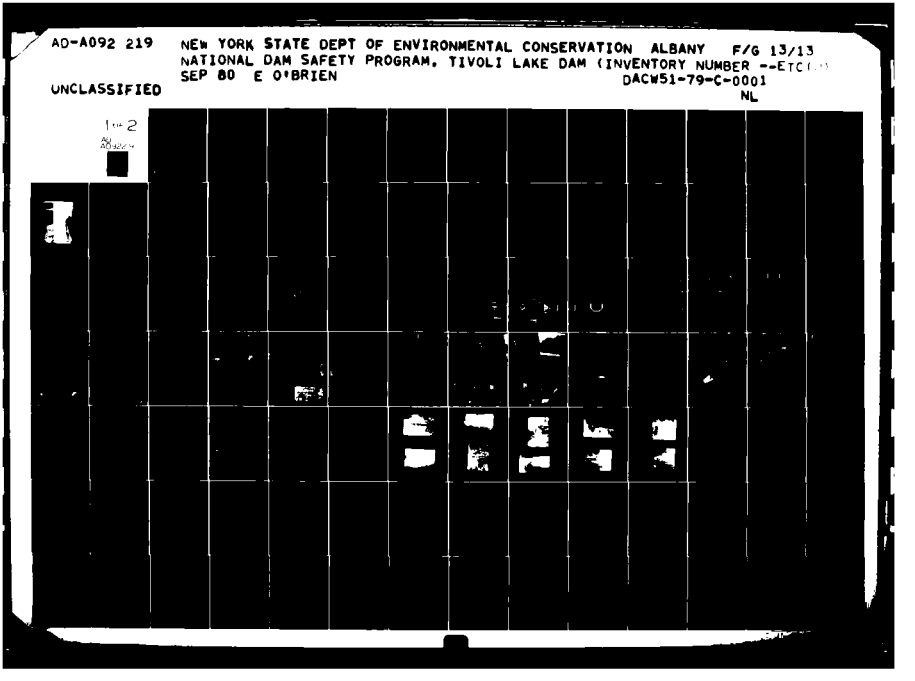

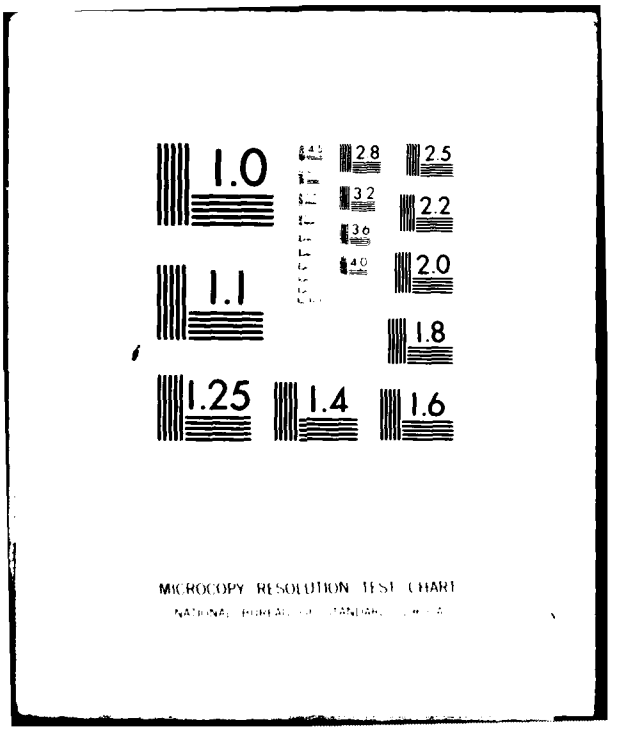
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Tivoli Lake
Rockland County
Ramapo River

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.

The examination of documents and the visual inspection of Tivoli Lake (Cranberry Lake) did not reveal conditions which constitute an immediate hazard to human life or property. However, the dam has some deficiencies which require further investigations and remedial actions.

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Using Corps of Engineers screening criteria for the initial review of spillway adequacy, it has been determined that the dam would be overtopped for all storms exceeding approximately 38 percent of the Probable Maximum Flood (PMF). The spillway is, therefore, adjudged as seriously inadequate and the dam is assessed as unsafe, non-emergency.

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean, however, that based on an initial screening, and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

In addition, a serious conflict exists with regard to actual elevations and those shown on the project drawings, making an accurate spillway adequacy analysis extremely difficult.

On the basis of structural stability performed during the investigation, the structural stability of the dam against overturning and sliding was determined to be adequate.

It is therefore recommended that within 3 months of notification [illegible] detailed hydrological and hydraulic investigations [illegible] verifying project elevation data [illegible] be undertaken to more accurately determine the [illegible] characteristics of the watershed and their affect upon the overtopping potential of the dam. The results of these investigations will determine the appropriate remedial measures which will be required to achieve a spillway capacity [illegible] to discharge without flow from at least the one-half PMF. In the interim, a detailed emergency action plan must be developed and implemented during periods of unusually heavy precipitation. Also, around-the-clock surveillance of the structure must be provided during these periods.

In addition, the dam has a number of problem areas which, if left uncorrected, have the potential for the development of hazardous conditions and must be corrected within one year:

1. Repair the 36-inch reservoir drain and gate operating mechanism.

2. Install an upstream gate and operating mechanism for the 10-inch reservoir drain.

3. Monitor the seepage in the right abutment of the dam with the aid of weirs.

4. Repair the cracking of the buttresses and decking and stop seepage emanating from cracks.

5. Repair spalled areas at the crest and downstream face of the spillway.

6. Clear away vegetation and maintain clear the areas adjacent to and downstream of the spillway.

7. Remove sediment build up upstream of the spillway.

8. Trim all vegetation on the abutments downstream of the dam and provide a program of periodic cutting.

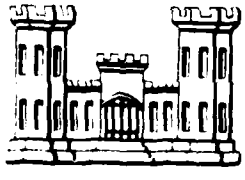
9. Provide a program of periodic inspection and maintenance of the dam and appurtenances, including yearly operation and lubrication of the reservoir drain system. Document this information for future reference. The aforementioned emergency action plan should be maintained and updated periodically during the life of the structure.

HUDSON RIVER BASIN

TIVOLI LAKE DAM

ROCKLAND COUNTY, NEW YORK
INVENTORY NO. N.Y. 52

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM



NEW YORK DISTRICT CORPS OF ENGINEERS

AUGUST 1980

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HUDSON RIVER BASIN

National Dam Safety Program

TIVOLI LAKE DAM

ROCKLAND COUNTY, NEW YORK

(INVENTORY NO. (Number) N.Y. 52),

Hudson River Basin

Rockland County New York.

PHASE I INSPECTION REPORT,

NATIONAL DAM SAFETY PROGRAM

(10) Eugene/Brien

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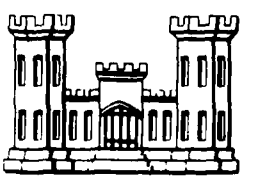

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NEW YORK DISTRICT CORPS OF ENGINEERS

AUGUST 1980

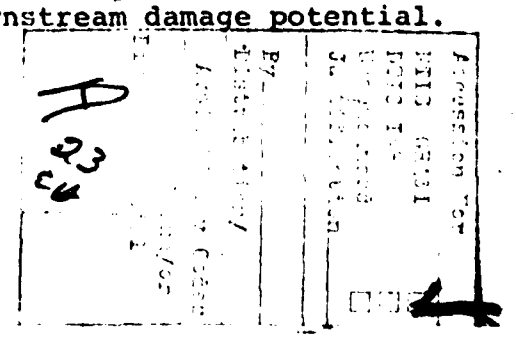
PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C., 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigations, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.



PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
TIVOLI LAKE DAM
I.D. NO. N.Y. 52
D.E.C. NO. 306
HUDSON RIVER BASIN
ROCKLAND COUNTY, NEW YORK

CONTENTS

APPENDICES

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
TIVOLI LAKE DAM
I.D. NO. N.Y. 52
D.E.C. NO. 306
HUDSON RIVER BASIN
ROCKLAND COUNTY, NEW YORK

Name of Dam:	Tivoli Lake (I.D. No. N.Y. 52)
State Located:	New York
County Located:	Rockland
Stream:	Ramapo River
Basin:	Hudson River
Date of Inspection:	April 24, 1980

ASSESSMENT

The examination of documents and the visual inspection of Tivoli Lake (Cranberry Lake) did not reveal conditions which constitute an immediate hazard to human life or property. However, the dam has some deficiencies which require further investigations and remedial actions.

Using Corps of Engineers screening criteria for the initial review of spillway adequacy, it has been determined that the dam would be overtopped for all storms exceeding approximately 38 percent of the Probable Maximum Flood (PMF). The spillway is, therefore, adjudged as seriously inadequate and the dam is assessed as unsafe, non-emergency.

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean, however, that based on an initial screening, and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

In addition, a serious conflict exists with regard to actual elevations and those shown on the project drawings, making an accurate spillway adequacy analysis extremely difficult.

On the basis of structural stability performed during the investigation, the structural stability of the dam against overturning and sliding was determined to be adequate.

It is therefore recommended that within 3 months of notification of the owner, detailed hydrological and hydraulic investigations and a resolution of conflicting project elevation datum of the structure be undertaken to more accurately determine the site specific characteristics of the watershed and their affect upon the overtopping potential of the dam. The results of these investigations will determine the appropriate remedial measures which will be required to achieve a spillway capacity adequate to discharge the outflow from at least the one-half PMF. In the interim, a detailed emergency action plan must be developed and implemented during periods of unusually heavy precipitation. Also, around-the-clock surveillance of the structure must be provided during these periods.

In addition, the dam has a number of problem areas which, if left uncorrected, have the potential for the development of hazardous conditions and must be corrected within one year:

1. Repair the 36-inch reservoir drain and gate operating mechanism.

2. Install an upstream gate and operating mechanism for the 10-inch reservoir drain.

3. Monitor the seepage in the right abutment of the dam with the aid of weirs.

4. Repair the cracking of the buttresses and decking and stop seepage emanating from cracks.

5. Repair spalled areas at the crest and downstream face of the spillway.

6. Clear away vegetation and maintain clear the areas adjacent to and downstream of the spillway.

7. Remove sediment build up upstream of the spillway.

8. Trim all vegetation on the abutments downstream of the dam and provide a program of periodic cutting.

9. Provide a program of periodic inspection and maintenance of the dam and appurtenances, including yearly operation and lubrication of the reservoir drain system. Document this information for future reference. The aforementioned emergency action plan should be maintained and updated periodically during the life of the structure.

Approved by:

Eugene O'Brien, P.E.
New York No. 29823

Col. W. M. Smith, Jr.
New York District Engineer

Date: 10 SEP 80

1. OVERVIEW OF DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
TIVOLI LAKE DAM
I.D. NO. N.Y. 52
D.E.C. NO. 306
HUDSON RIVER BASIN
ROCKLAND COUNTY, NEW YORK

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority

The Phase I inspection reported herein was authorized by the State of New York, Department of Environmental Conservation by a letter dated 7 January 1980, in fulfillment of the requirements of the National Dam Inspection Act, Public Law 92-367, 8 August 1972.

b. Purpose of Inspection

This inspection was conducted to evaluate the existing conditions of the dam, to identify deficiencies and hazardous conditions, to determine if they constitute hazards to human life and property, and to recommend measures where required.

1.2 DESCRIPTION OF THE PROJECT

a. Description of the Dam and Appurtenant Structures

The Tivoli Lake Dam (Cranberry Lake) is composed of a 140 foot long, 25 foot high Ambursen type buttress dam and a 165 foot long broad crested concrete spillway. A 36-inch diameter concrete conduit and a 10-inch diameter steel pipe located in the base of the dam serve as reservoir drains; the flow through these pipes is regulated through different valves at separate locations.

This Ambursen type (reinforced concrete flat slab and buttress) dam has an upstream face with a 1V to 1.5H slope and an "open" downstream side. The crest width of the dam is 2.5 feet and the base width is 45 feet. The dam is made up of seven buttresses and two end walls which form 9 bays - 7 center bays are 15 feet and either end bay is 14.4 feet wide. Rectangular reinforced concrete beams of varying dimensions support the 9-inch thick reinforced concrete deck. There is also an approximately 150 foot long wing wall on the left abutment beginning at the end wall of the dam.

There are two reservoir drains which pass beneath the dam. The larger of the two, a 36-inch diameter concrete conduit is controlled by a vertical slide gate operated from a bridge extending from the crest of the dam approximately

40 feet upstream. The outlet for this pipe is located approximately 15 feet downstream of the downstream edge of the buttresses. The second reservoir drain is a 10-inch diameter steel pipe with its intake immediately adjacent to, and at the same level as, the 36-inch drain. The control for this pipe is a gate valve located in the bay to the left of the center near the downstream edge of the buttress. The outlet for this pipe is located in a stream about 200 feet downstream of the dam.

The spillway is a broad crested (0.5 to 1.0 feet wide) reinforced concrete overflow structure located about 0.5 mile northeast of the dam. The spillway had previously been reported to be 150 feet long. Additions were made near the right abutment in the 1970's to upgrade the spillway increasing its length to 165 feet.

b. Location

Tivoli Lake Dam (presently known as Cranberry Lake Dam) is located on the Ramapo River, a tributary of the Hudson River, southwest of Route 17 near Sloatsburg, New York.

c. Size Classification

The dam has a structural height of 29 feet, and a storage capacity of less than 1,000 acre-feet. Therefore, it is classified as a small dam.

d. Hazard Classification

The dam is in the high hazard category. There are a few homes and other structures, as well as the town of Sloatsburg about one mile downstream from the dam.

e. Ownership

Tivoli Lake Dam (Cranberry Lake Dam) is owned by the Ramapo Land Company, P.O. Box 45, Sloatsburg, New York, 10974, Tel. (914) 753-5228. The person to contact is Mr. Vanderhoes.

f. Purpose of Dam

The dam impounds water for recreational use of the owner(s).

g. Design and Construction History

The dam and its appurtenant structures were designed by Clark and Company of New York between 1905 and 1910. The construction of the dam was completed in 1908 and the name of the contractor is unknown.

h. Normal Operating Procedures

There is no established operating procedure for the outlets on Tivoli Dam. The gate control for the 36-inch drain has been inoperable for at least 25 years according to Mr. Anthony Spadavecchia, Superindendent for the Ramapo Land Company. The

valve for the 10-inch drain is opened periodically to check its operability.

1.3 PERTINENT DATA

a. Drainage Area (sq. mile)	2.98
b. Discharge at Dam Site (cfs)	
Ungated Spillway at Maximum Pool	3350
Regulating Outlets at Maximum Pool	200
c. Elevation (feet above MSL Datum)	
Top of Dam	512.5
Maximum Design Pool	509.0
Spillway Crest	509.0
Invert 36-Inch Reservoir Drain	488.3
Invert 10-Inch Reservoir Drain	488.3
d. Reservoir	
Length of Maximum Pool (miles)	2.5
Length of Shoreline (miles)	0.47
Surface Area (acres)	104.8
e. Storage (acre-feet)	
Spillway Crest	790
Top of Dam	1170
f. Dam	
Type of Dam	Ambursen
Length	140 feet
Upstream Slope	1V:1.5H
Downstream Slope	Open Face
Crest Width	2.5 feet
Grout Curtain	None
g. Spillway	
Type	Broad crested overflow
Length	165 feet
Upstream Channel	None
Downstream Channel	Natural - "Boulder Paved"
h. Regulating Outlets	
Intake (1)	Upstream Toe - El. 488.3
Intake (2)	Upstream Toe - El. 488.3
Control (1)	Slide Gate - Controlled from Bridge
Control (2)	Valve - Controlled Downstream of Dam
Pipe (1)	36-Inch Concrete Conduit
Pipe (2)	10-Inch Steel Pipe
Outlet (1)	15 feet Downstream of Buttress
Outlet (2)	150 feet Downstream of Buttress

SECTION 2 - ENGINEERING DATA

2.1 GEOLOGY

The bedrock in the area of Tivoli Lake Dam and Reservoir is composed of Precambrian formations: interlayered hornblende granite gneiss and amphibolite on the southeastern half of the reservoir and quartz plagioclase gneiss in the remainder of the reservoir. The gneiss is exposed in several areas around the reservoir, in particular, on the left abutment of the spillway. The gneiss appears to be competent although some surface cracking appears in the outcrops. Tivoli Lake is located approximately five miles from the Ramapo fault along which there has been some recent seismic activities.

2.2 SUBSURFACE INVESTIGATIONS

The files of the Ramapo Land Company contain no data on site geology and foundation conditions. The search in connection with this inspection did not reveal any information on exploratory borings or foundation investigations made prior to or during construction. Some shallow test pits were reportedly dug in the right abutment recently but data for these pits were not available. However, there are data available in the literature on the general soil conditions of the area. Surface cover in the vicinity of Tivoli Lake Dam is described as "Rockland" (Ref. 9). Rock outcrops, stones and steep slopes are characteristic of this area of very thin glacial till over bedrock.

2.3 DAM AND APPURTENANT STRUCTURES

The dam was designed by Clark and Company of New York (between 1905-1910) and some plans, sections and details of the dam, outlet works and spillway are available. There are, however, no design calculations, as-built drawings or construction records for the dam or other structures. Additionally, there are sketches of the dam and spillway done in 1920 for repairs to the deck in certain places. Also, there are sketches for deck surface patching carried out in the mid-1970's and proposed spillway improvements much of which was not constructed (See Appendix A).

2.4 CONSTRUCTION RECORDS

No construction records have been located for the project.

2.5 OPERATION RECORDS

There is no operation record or manual for the project. There is no regular operation of outlet works. Since about 1970, on behalf of the owner, the dam has been visually inspected

annually by a professional engineer and a report with recommendations has been given to the owner. Reports of inspection are available at the office of the owner. Copies of reports are given in Appendix F. There are records of certain repairs of spillway and dam at the office of the owner. No other systematic monitoring of the performance of the dam is in effect.

2.6 EVALUATION OF DATA

The existing data has been made available by the owner at his office, including the inspection reports of his engineers. The drawings provide basic information of the as-designed structural dimensions of the dam, outlet works and spillway. They provide no information on the nature of the foundation and probably do not represent as-built conditions. The elevations shown on the design drawings do not agree with those shown on the current USGS sheet. As an example, on the original drawings, crest elevation of the dam is shown at about 469. It is quite apparent from the USGS sheet that the elevation of the crest of the dam is approximately 512.5. Inquiries to the owner and the county drainage engineer could not resolve the conflict. To facilitate the use of existing hydrologic and hydraulic data as required in a Phase I inspection, the USGS elevation have been utilized. However, the information available appears to be adequate for the purpose of the Phase I inspection.

SECTION 3 - VISUAL INSPECTION

3.1 VISUAL INSPECTION

a. General

The visual inspection of Tivoli Lake Dam (Cranberry Lake Dam) and its appurtenant structures was carried out on April 24, 1980. The weather was fair and the temperature in the 60 to 65°F range. The reservoir was at what is described as normal "full" capacity (water just passing over the spillway).

b. Main Dam

The buttresses, wing walls and deck plates appear to be in generally good condition. There is evidence of minor cracking and spalling of the concrete in the buttresses and the deck, but much of this has been repaired and is in good condition. The vertical and horizontal alignment of the crest appears to be unchanged. There are no open cracks in the concrete which would indicate movement of the dam. However, the following adverse conditions were noted.

(1) There appears to be some cracking of the buttress between bays 8 and 9. A small amount of water flows continuously from this crack. Additionally, minor structural cracking and leakage exists in the buttresses between bays 5-6 and 1-2 and in the deck plates of bays 2 and 5 (See Appendix A - Drawings for location of bays).

(2) The concrete decking of the dam has minor leaks in many areas including areas where repairs have been previously carried out.

(3) The right abutment directly downstream of the dam is soaked, wet and covered with a large amount of vegetation. Springs appear on the slope and there is evidence of sloughing and sliding.

(4) French drains have been installed in the right abutment to drain and stabilize the slope. Vegetation directly above these drains indicates that they are operating properly.

c. Spillway and Tailrace

The spillway and tailrace area are generally in good condition. There are, however, some adverse conditions which need attention during normal maintenance. Included in these are: heavy sedimentation upstream of the spillway (to about 6 inches below the spillway crest), minor cracks and spalls in the concrete which will continue to grow, and heavy vegetation and trees in the channel downstream of the spillway.

d. Regulating Outlets

The 10-inch reservoir drain appears to be in good operating condition. There are, however, several problems with the 36-inch drain which present possible future problems and should receive attention:

(1) The control and slide gate for the outlet are inoperable due to the lack of the required access bridge, and a connecting rod required for the gate has corroded away.

(2) There is a slight leakage around the seal of the 36-inch slide gate and through spalls in the concrete conduit resulting in a small but constant flow through the pipe.

e. Reservoir Area

There were neither slides, rockfalls, sloughing or other signs of instability noted in the reservoir with the exception of those already discussed on the right abutment, nor were objectionable amounts of floating debris observed in the reservoir.

3.2 EVALUATION OF OBSERVATIONS

Although deficiencies were observed, there is no indication that the dam is in imminent danger. A number of the deficiencies listed in the previous paragraph are minor and may be corrected by maintenance forces. Significant conditions were observed, however, which require investigation to insure the stability of the dam and appurtenances. The following is a summary of the problem areas encountered, in order of importance, with the appropriate recommended action:

(1) The operating mechanism of the gate valve of the 36-inch reservoir drain has corroded away and the gate is stuck in the closed position. The operating mechanism should be repaired.

(2) The right abutment area downstream of the dam is saturated and seepage is evident. This seepage should be monitored with the aid of weirs. If the flow rate increases significantly, or the migration of fines occurs, immediate remedial measures will be required to control the seepage.

(3) The deck and buttresses of the dam exhibit cracking and leakage. These cracks should be patched and the leakage stopped. If leakage continues, more extensive repairs may be required.

(4) There are spalled areas at joints along the crest and downstream face of the spillway. These should be patched to prevent further deterioration.

(5) Remove vegetation from and keep vegetation clear from the areas adjacent to and downstream of the spillway.

(6) Remove sediment which has built up upstream of the spillway.

(7) Cut all vegetation on the abutments downstream of the dam and provide a program of periodic cutting.

SECTION 4 - OPERATION AND MAINTENANCE PROCEDURES

4.1 PROCEDURES

There are no regulating or operating procedures for the spillway or reservoir drains. The 10-inch diameter reservoir drain is periodically opened to check its operability. The spillway is uncontrolled.

4.2 MAINTENANCE OF DAM

Over the past ten years, and currently, the dam, spillway and appurtenant structures have been inspected annually and reports have been filed containing recommendations for repairs and improvements. Several repair programs have been carried out to improve the concrete deck plates, buttresses and spillway surface. It appears from observations that these repairs have been satisfactory and have improved the structures. In addition, repair work including excavation of deteriorated materials, backfilling with impervious materials and covering with riprap has been carried out to correct erosion, seepage and slope stability problems at the abutments, particularly on the right abutment.

The 36-inch diameter gate valve for the reservoir is not in operating condition. For several years, the gate has been inoperable due to a missing connection stem from the surface to the gate valve and an access bridge no longer exists. The 10-inch diameter valve is in operating condition and its condition is checked periodically.

4.3 WARNING SYSTEM IN EFFECT

There is no warning system in effect or in preparation.

4.4 EVALUATION

The dam has generally been maintained in operating condition except for minor vegetative growth, minor seepage and the 36-inch reservoir drain, which is inoperable.

SECTION 5. - HYDROLOGIC/HYDRAULIC

5.1 DRAINAGE BASIN CHARACTERISTICS

Tivoli Lake Dam is located southwest of Sloatsburg in Rockland County, New York. The drainage area contributing to Tivoli Lake (Cranberry Pond) is 2.98 square miles, and consists of fairly steep slopes with relatively wide valleys, heavily wooded, with sparse development. Some storage is available in the basin, in the form of an upstream pond (Potake Pond) and some small swamps. The lake occupies 104.8 acres or 5.5 percent of the drainage basin.

5.2 ANALYSIS CRITERIA

The hydrologic/hydraulic analysis of the Tivoli Lake Dam was performed using the U.S. Army Corps of Engineers HEC-1 Program (Ref. 1). The unit hydrograph used in this analysis was transposed from a nearby basin. The original unit hydrograph was developed for a sub-area of the Ramapo River basin in a previous study (Ref. 2). In accordance with the recommended guidelines of the Corps of Engineers (Ref. 7), the adequacy of the spillway was analyzed using the Probable Maximum Flood (PMF) and one-half the PMF. The storage capacity of Potake Pond was not included in this analysis.

5.3 SPILLWAY CAPACITY

The principal spillway of Tivoli Lake is located approximately 2,000 feet northwest of the dam and consists of a 150 foot long uncontrolled concrete wall, with a crest width of about 6 inches. There are essentially no training walls on either side of the spillway, therefore, a minimal rise in the water surface would cause flow over natural ground. At the northern end of the spillway, the ground is steep, while at the other end of the rise is much gentler and slopes upward at a slope of approximately 20.0 percent. It is estimated that with the water surface at El 512.5 (equivalent to the top of the dam), the discharge over the spillway and bank would be 3,350 cfs.

5.4 RESERVOIR CAPACITY

Normal capacity of Tivoli Lake, at El 509 (equivalent to spillway crest elevation) is reported to be 790 acre-feet (Ref. 5). Additional surcharge storage capacity to the top of the dam, which was assumed to increase linearly, is 380 acre-feet and was computed from areas planimetered from the USGS

Quadrangle sheet, Sloatsburg. Therefore, the maximum reservoir capacity to the top of the dam, El 512.5 is 1,170 acre-feet.

5.5 FLOODS OF RECORD

No information was available regarding the occurrence or magnitude of floods at this location.

5.6 OVERTOPPING POTENTIAL

The potential of the dam being overtopped was investigated on the basis of the spillway discharge, capacity and the available surcharge storage being able to meet the selected design flood inflows. The peak inflow discharges computed were 9,463 cfs and 4,731 cfs for the PMF and one-half the PMF, respectively. The peak PMF outflow was 8,762 cfs, which overtopped the dam by 1.9 feet. The peak outflow for one-half the PMF was 3,907 cfs and overtopped the dam by 0.32 feet.

Using the Corps of Engineers screening criteria (Ref. 7) for initial review of the adequacy of the spillway, it is estimated that the dam would be overtopped by any flood exceeding 38 percent of the PMF.

5.7 EVALUATION

The dam does not have sufficient spillway capacity to pass either the PMF or one-half the PMF without overtopping. The spillway is assessed as seriously inadequate because of the following conditions:

(1) Flow over the natural ground on the right abutment of the spillway could cause erosion and the possible failure of the spillway.

(2) Overtopping of the dam could result in erosion of the right abutment and the toe of the dam and lead to the potential instability of the dam.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

Visual observations did not indicate any serious structural problems with the dam or the spillway structures. The structural cracking in the buttress at bay 8-9 and the spalling and minor cracking of the plates and the spillway is not considered to represent an unstable or otherwise dangerous condition.

b. Design and Construction Data

There are no design computations or other data regarding the structural stability of the dam or spillway available. There are drawings of the original design available showing plans, sections and some details (See Appendix A).

c. Stability Analysis

The primary sources of structural and foundation condition information used in the analysis were as follows:

(1) The dimensions of the dam were taken from the available drawings which were spot-checked during the field inspection.

(2) The subsurface conditions were estimated on the basis of observations made in the field.

The following table shows the results of the structural stability analysis for the dam. The computations of the analysis is given in Appendix E.

CASE(S)	OVERTURNING	SLIDING FACTOR OF SAFETY
I) Normal Loading Condition with Reservoir Level at Spillway Crest; no ice load.	Within Middle Third	1.5
II) Normal Loading Condition with Reservoir Level at Spillway Crest; with ice load = 5 kips	Within Middle Third	1.2
III) Unusual Loading: 1/2 PMF, water flowing over Spillway at Depth of 4.12 feet and over dam at Depth of 0.62 feet	Within Middle Half	1.25

CASE(S)	OVERTURNING	SLIDING FACTOR OF SAFETY
IV) Extreme Loading: PMF, water flowing over spillway by 5.8 feet and over the top of the dam by 2.36 feet	Within Middle Half	1.05
V) Unusual Loading: Lake level at Spillway Crest and Earthquake force of 0.05	Within Middle Half	1.36

On the basis of the structural stability performed during the investigation, the structural stability of the dam against overturning was determined to be adequate for all cases. The stability of the dam against sliding was determined to be adequate for all cases except normal loading with ice load. However, in view of the sloping upstream concrete face of the dam, the ice will ride up the face of the dam and the dam will not be subjected to a full 5 kip ice loading. The factor of safety of 1.2 can therefore be considered adequate against sliding under ice loading.

d. Operating Records

There is no regular operation or operation records of the regulating gates. There have been no major structural problems since the 1940's when the deck was partially reconstructed to the original design reported. The non-operational 36-inch reservoir drain could effect the stability of the dam.

e. Post-Construction Changes

All reported post-construction work on the dam has been in the form of repairs and rebuilding without altering the original design. The spillway has been modified to increase its length to 165 feet.

f. Seismic Stability

The dam is located in Zone 2, therefore, a stability analysis was carried out using a normal reservoir loading (water level at spillway crest) and a 0.05g earthquake factor with Zanger's method. The results of this analysis showed the dam to be safe under both overturning and sliding.

SECTION 7 - ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety

The Phase I investigation of Tivoli Lake Dam revealed that the spillway is "seriously inadequate", based on the Corps of Engineers screening criteria, and outflows from any storm in excess of 38 percent of the PMF will overtop the dam. The overtopping of the dam could cause erosion of the abutments, undermining of the toe and subsequent failure of the dam. The resulting flood wave would significantly increase the hazard to downstream residents. For these reasons, the dam has been assessed as unsafe, non-emergency.

The classification of "unsafe" applied to a dam because of a "seriously inadequate" spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency in spillway computations, that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

On the basis of structural stability performed during the investigation, the structural stability of the dam against overturning was determined to be adequate for all cases. The stability of the dam against sliding was determined to be adequate for all cases except normal loading condition with ice load. The normal ice loading considered however is not justified for this dam due to its sloping upstream concrete face which will result in ice ride up and a smaller loading. The stability of the dam is therefore adjudged adequate for all cases.

In addition, the dam has a number of deficiencies which, if left uncorrected, have the potential for the development of hazardous conditions.

b. Adequacy of Information

The information reviewed is considered adequate for a Phase I investigation, except for the discrepancy of project datum as described in Section 2.6.

c. Need for Additional Investigations

Since the spillway is considered to be "seriously inadequate", additional hydrologic/hydraulic investigations are required to more accurately determine the site specific characteristics of the watershed. Included in these investigations should be a resolution of the discrepancy of the project datum. After the in-depth hydrologic/hydraulic investigations

have been completed, remedial measures must be initiated to provide spillway capacity sufficient to discharge the outflow from the one-half PMF event. At the same time, structural analysis should be performed to determine the level to which the reservoir must be lowered in the winter months to meet the sliding factor of safety (S.F.S. $\geq$ 1.5) under ice loading condition.

d. Urgency

The additional hydrologic/hydraulic investigations which are required must be initiated within 3 months from the date of notification, at the same time the discrepancy in project datum should be resolved. Within one year of notification, remedial measures as a result of these investigations must be initiated with completion of these measures during the following year. In the interim, develop an emergency action plan for the notification of downstream residents and proper governmental authorities in the event of overtopping and provide around-the-clock surveillance of the dam during periods of extreme runoff. The other problem areas listed below must be corrected within one year from notification.

7.2 RECOMMENDED MEASURES

1. Results of the aforementioned investigations will determine the type and extent of remedial measures required.

2. The operating mechanism for the 36-inch reservoir drain should be made operable.

3. An upstream control should be provided for the 10-inch reservoir drain.

4. The seepage occurring in the right abutment should be monitored and observations recorded. Construction of weirs and monitoring of flow at biweekly intervals should be performed to properly ascertain the nature of the seepage.

5. The cracking of the buttresses and decking should be repaired and the seepage stopped.

6. Spalled areas and cracks in the concrete on the spillway should be repaired.

7. Remove vegetation from and keep vegetation clear from the areas adjacent to and downstream of the spillway.

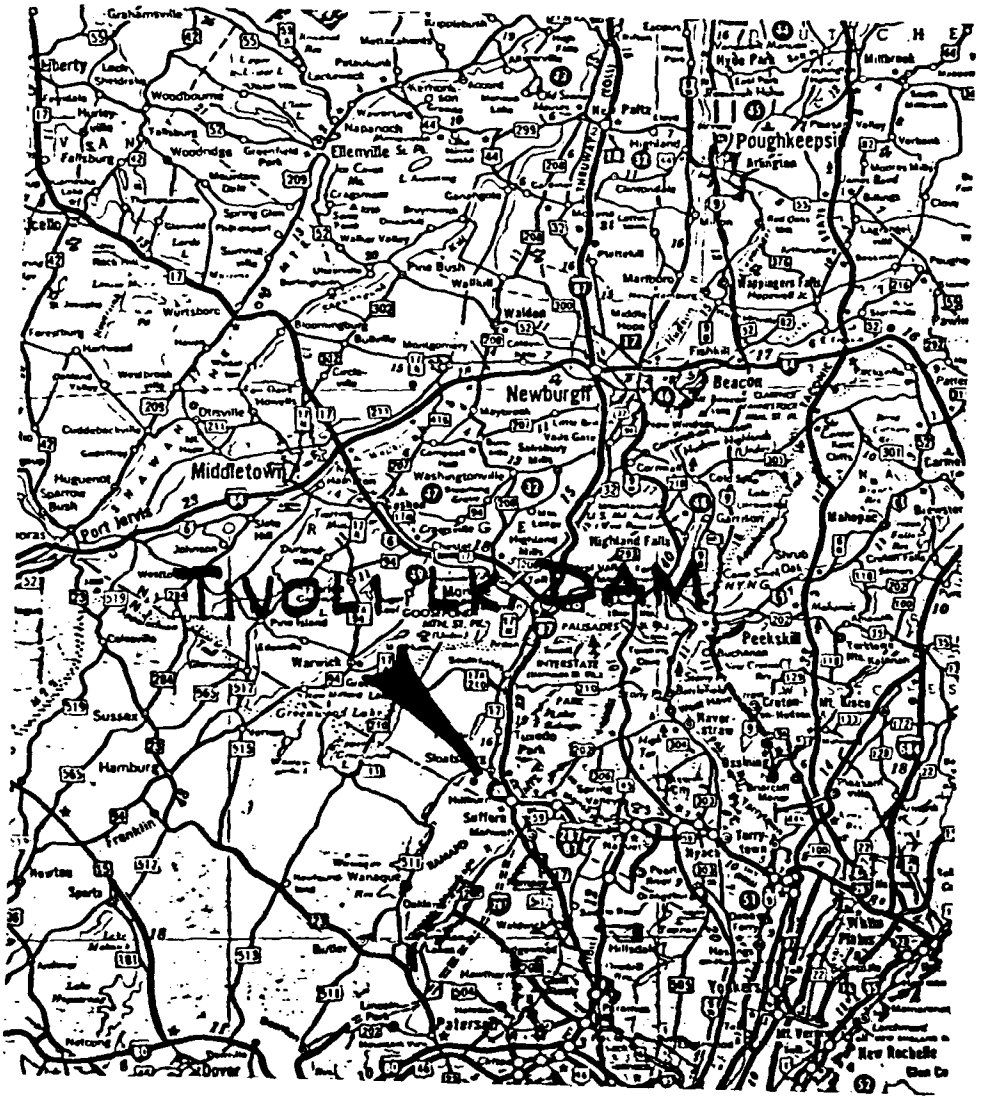
8. Remove sediment which has built up upstream of the spillway.

9. Cut all vegetation on the abutments downstream of the dam and provide a program of periodic cutting.

10. Provide a program of periodic inspection and maintenance of the dam and appurtenances including yearly operation and lubrication of the reservoir drain system. Document this information for future reference. The emergency action plan described in Section 7.1d should be maintained and updated periodically during the life of the structure.

DRAWINGS

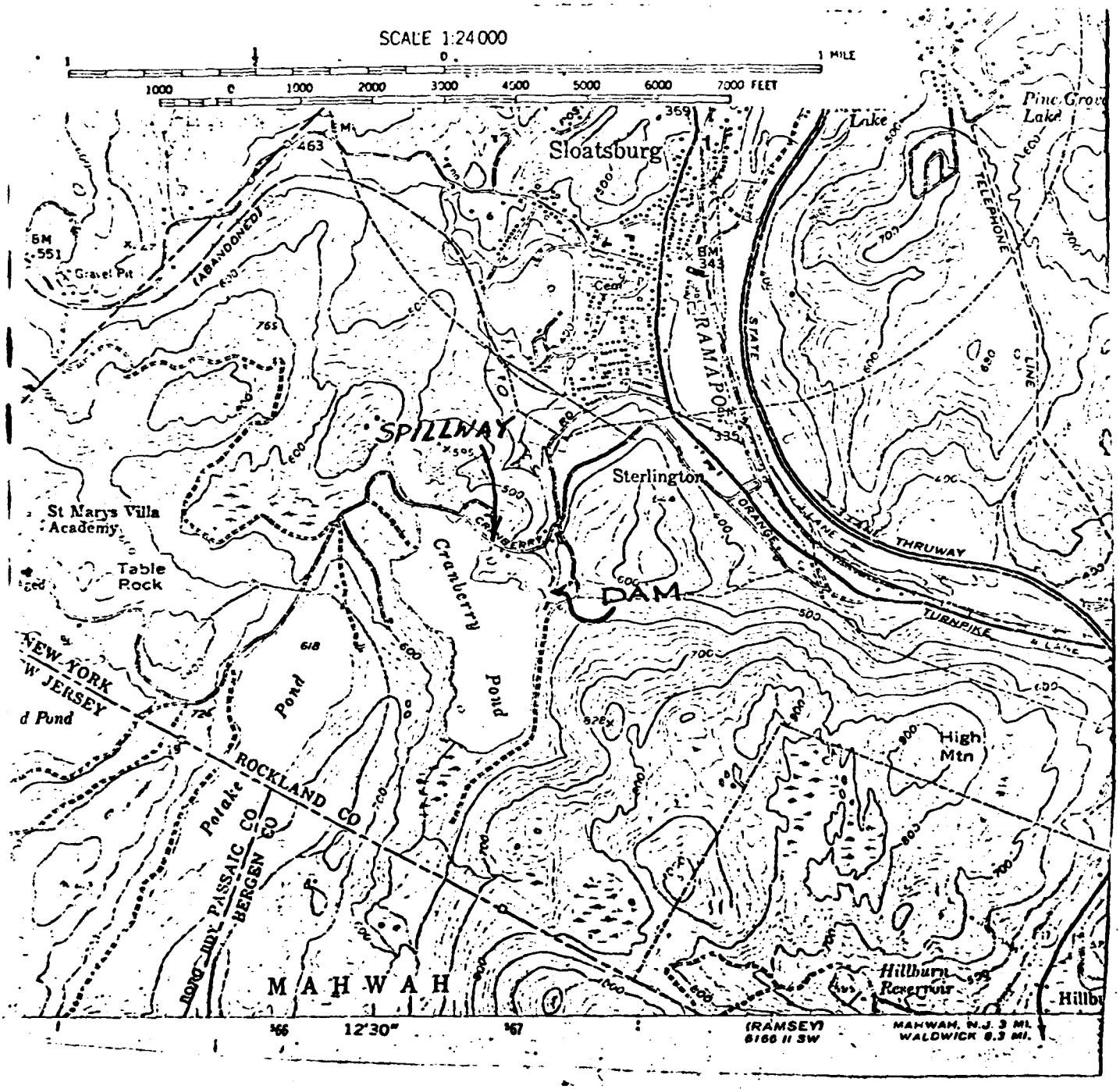
APPENDIX A



SCALE: 1 Inch = 11.2 Miles

VICINITY MAP
TIVOLI LAKE DAM

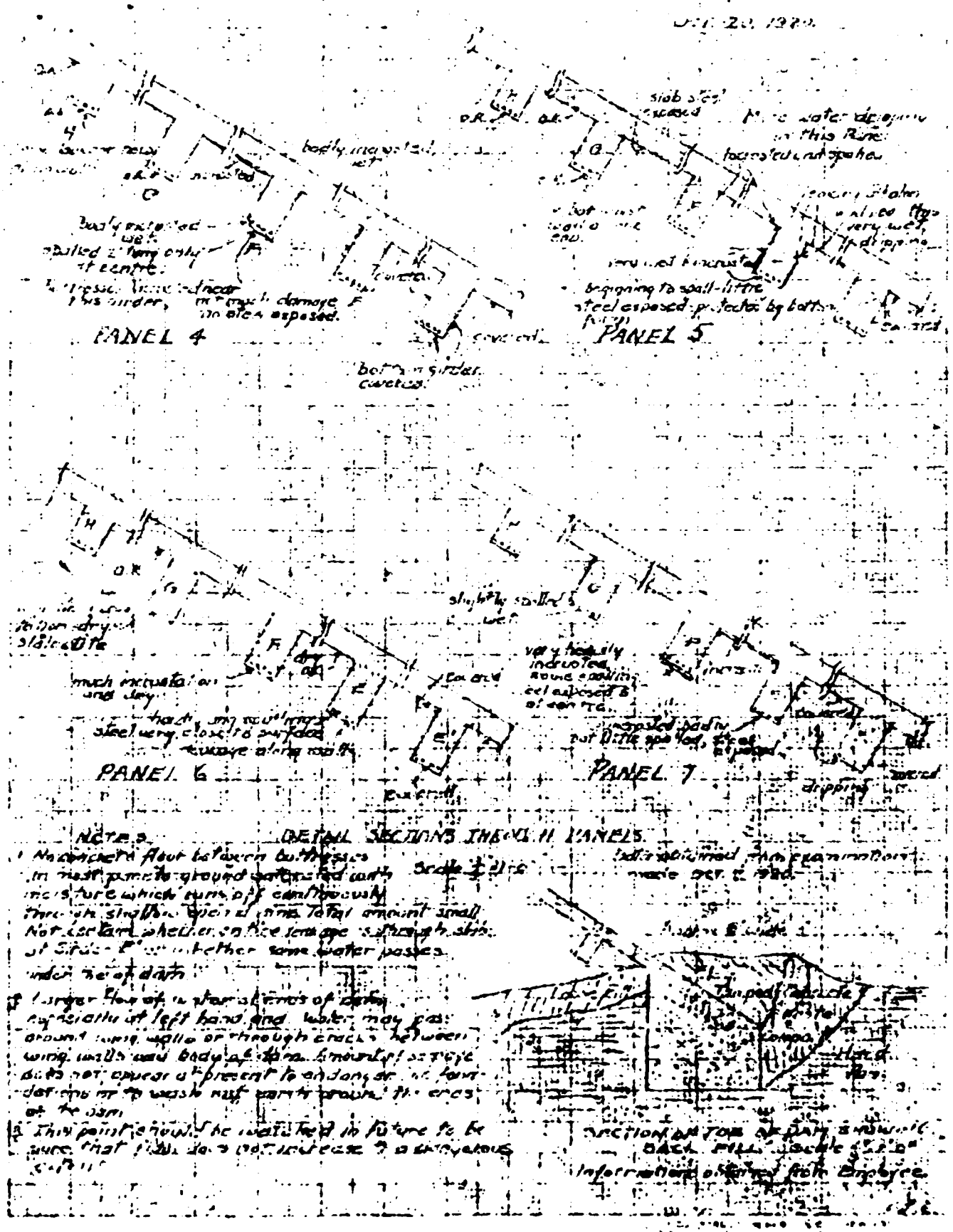
SLOATSBURG QUAD
New York - New Jersey

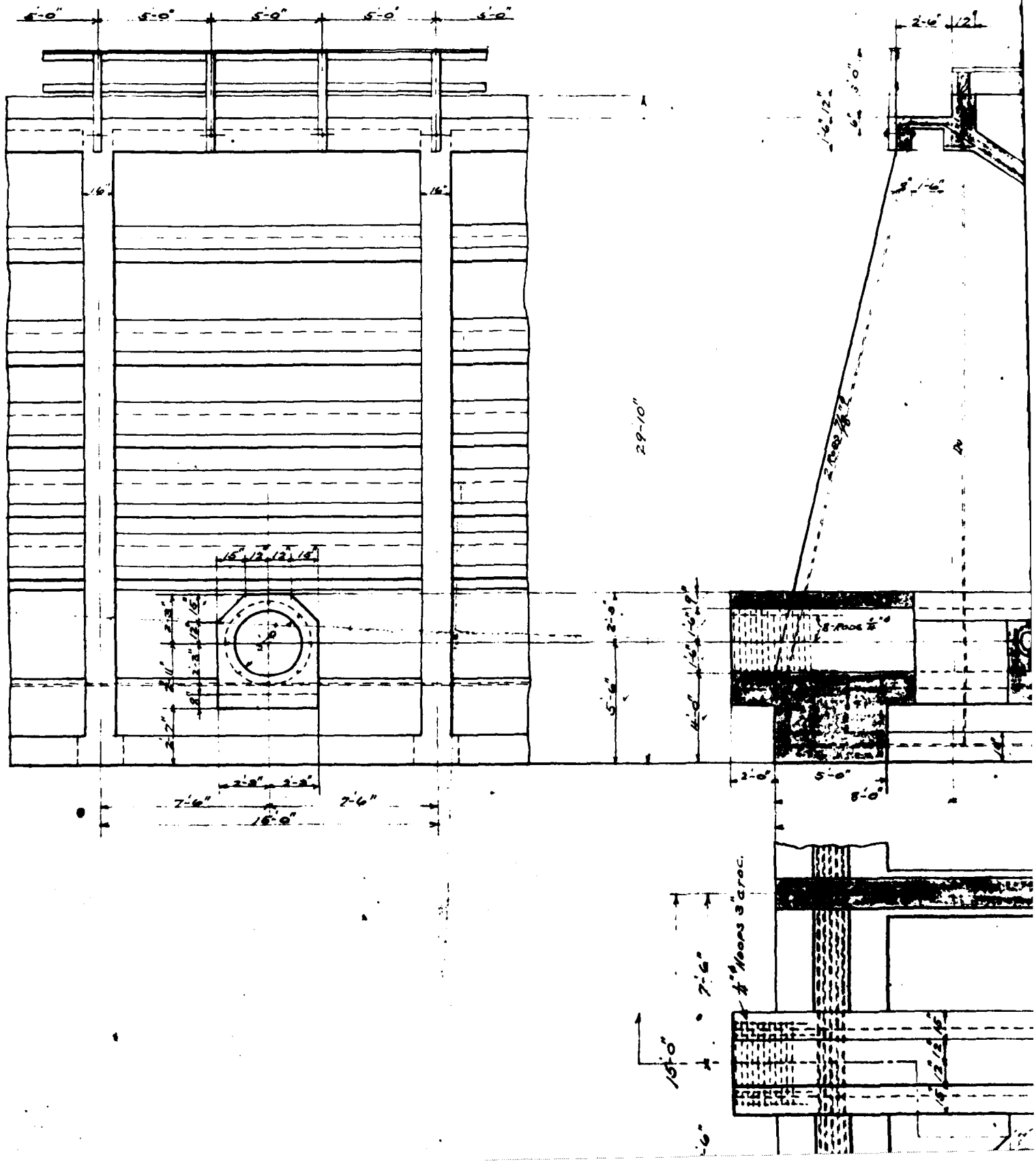


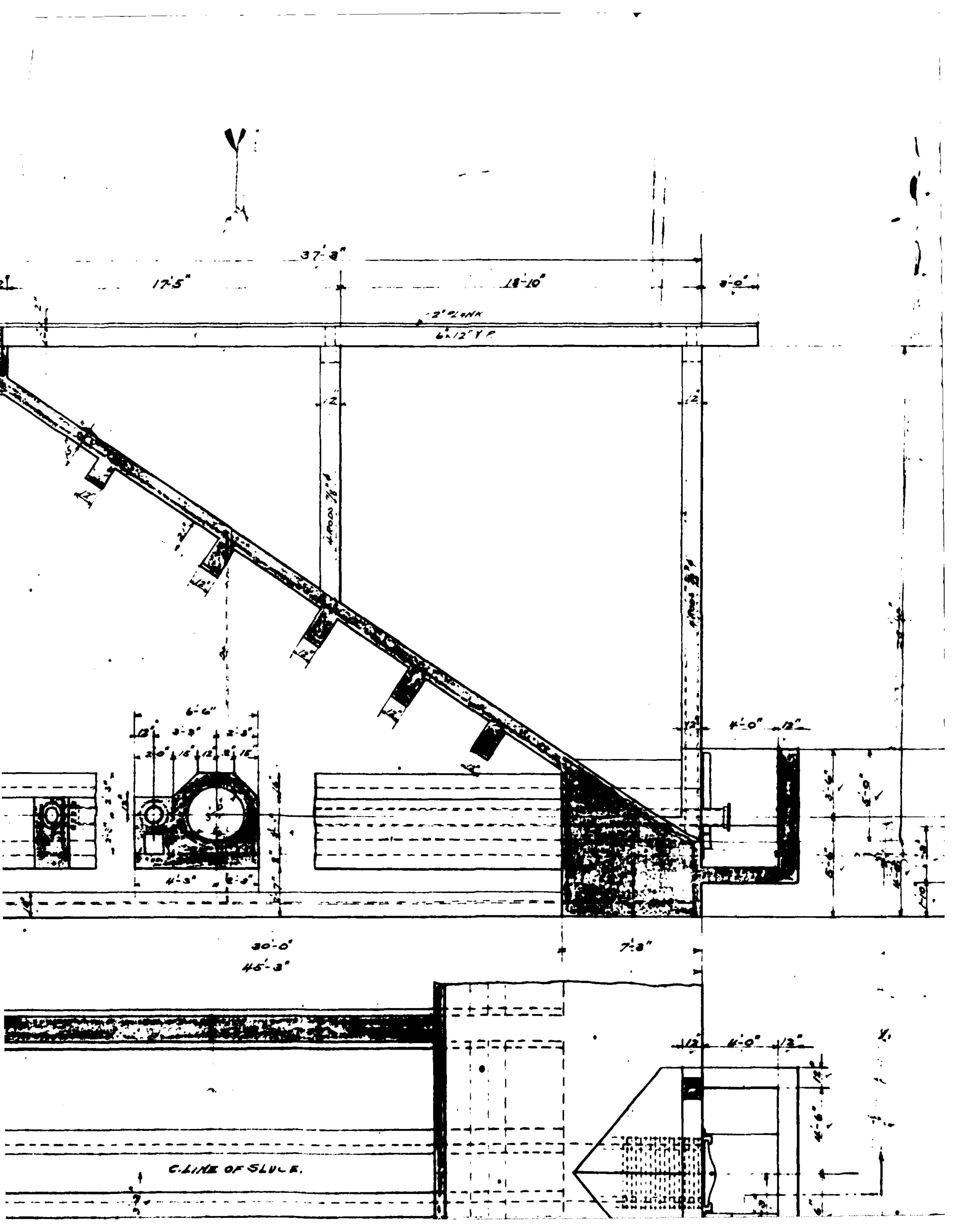
TOPOGRAPHIC MAP
TIVOLI LAKE DAM

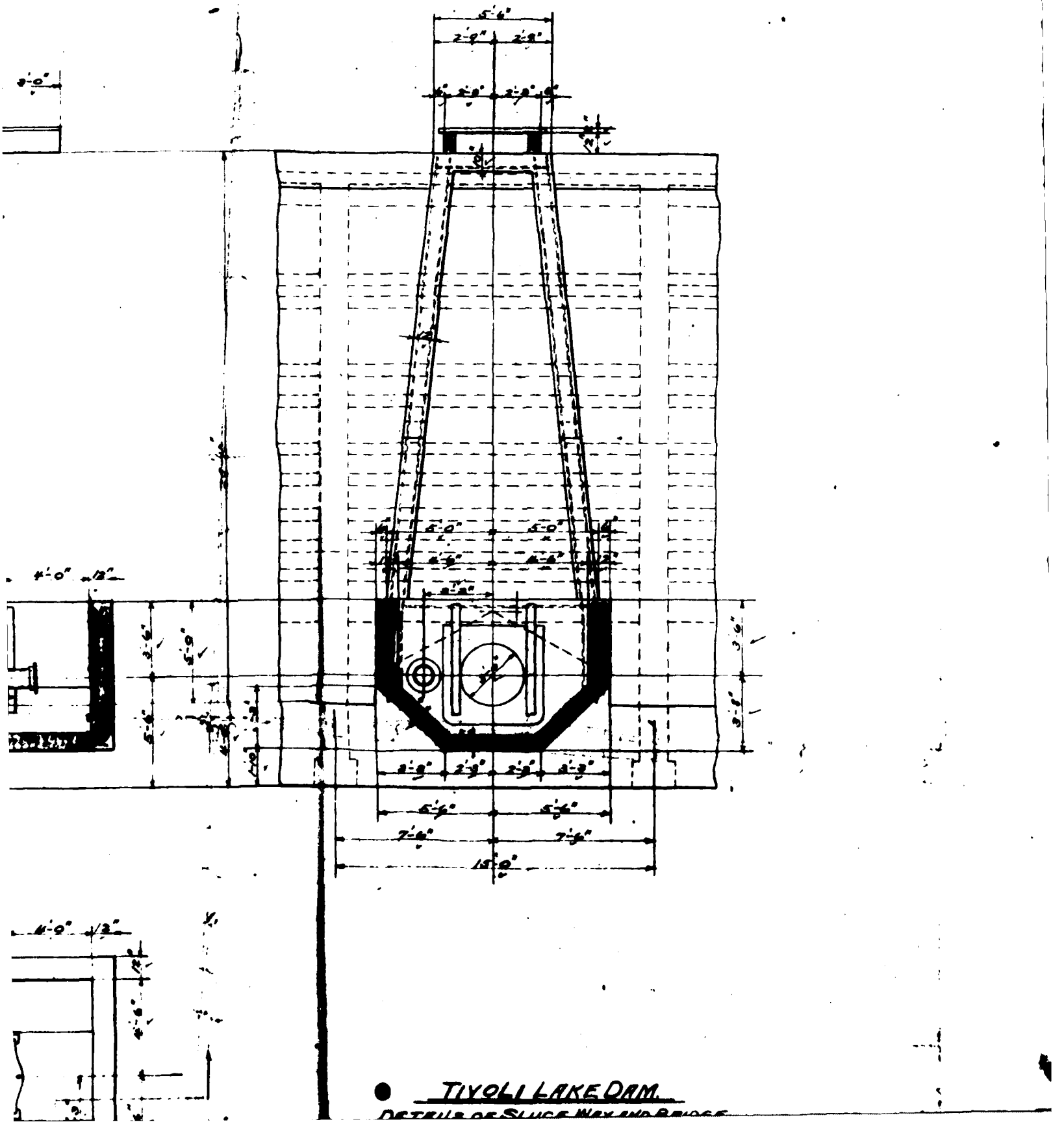
TIVOLI LAKE DAM NEAR STEPHENTOWN

OCT. 20, 1939



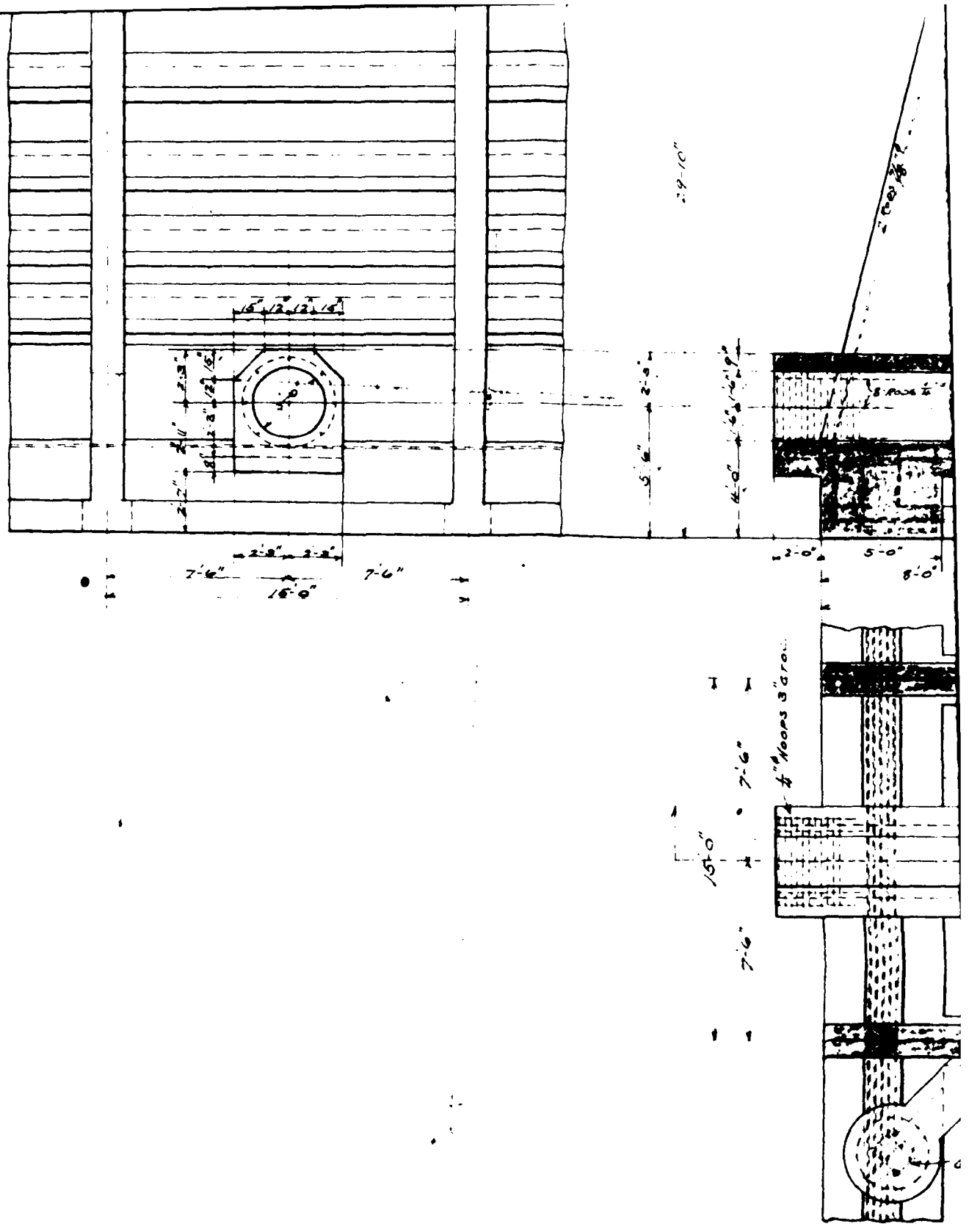

5'-0"
5'-0"
5'-0"
5'-0"
5'-0"
16"
16"
15" 1/2" 12" 15"
2'-2"
2'-2"
2'-2"
7'-6"
2'-6"
16'-0"
29-10"
2'-6" 1/2"
5'-6"
2'-0"
5'-0"
8'-0"
#4 Hoops 3" oc.
7'-6"
13'-0"

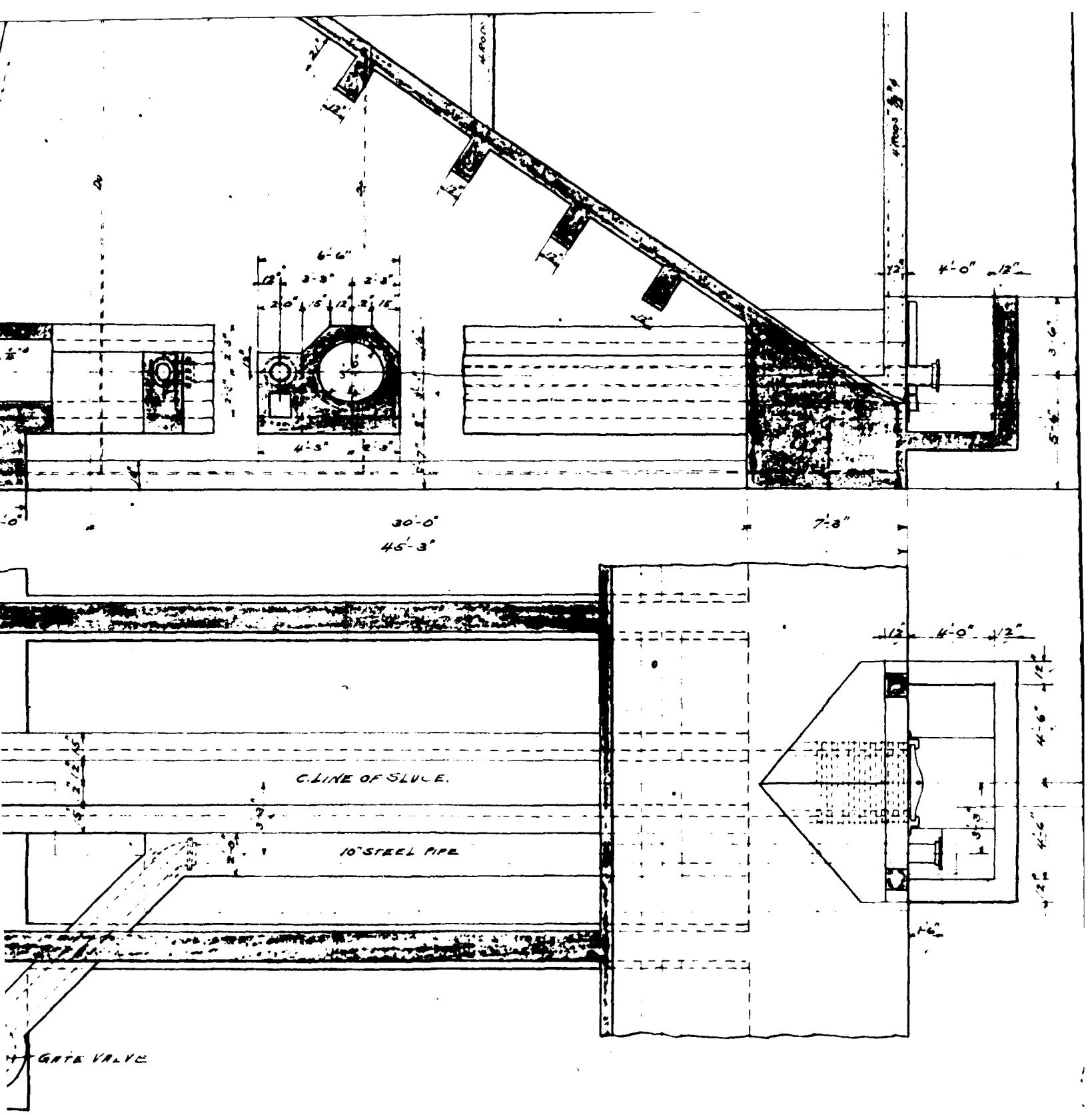

37'-3"
17'5"
18'-10"
3'-0"
2" PLANK
6"x12" Y.P.
12"
4 RODS 7/8"
6'-6"
12"
3'-3"
2'-3"
2'-0"
15"
4'-3"
2'-3"
4'-0"
12"
30'-0"
45'-3"
7'-3"
4'-0"
4'-6"
C. LINE OF SLUICE.

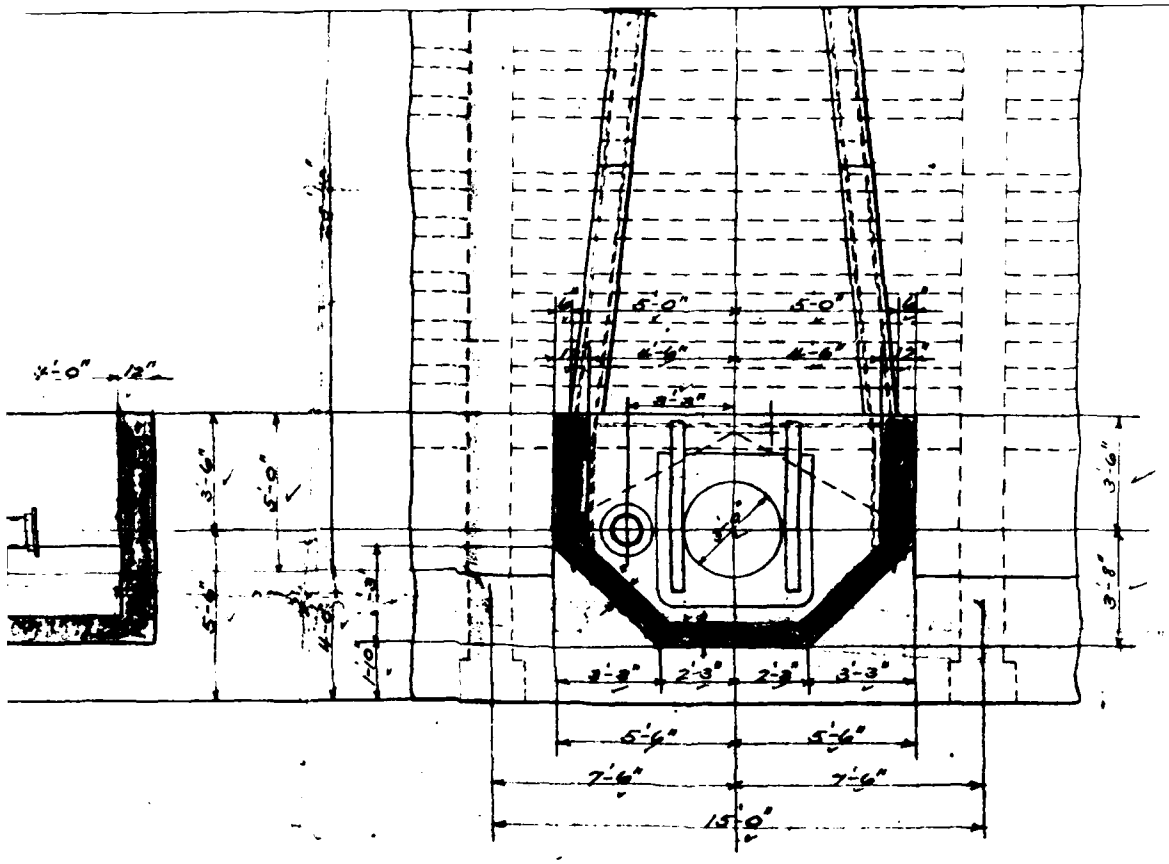


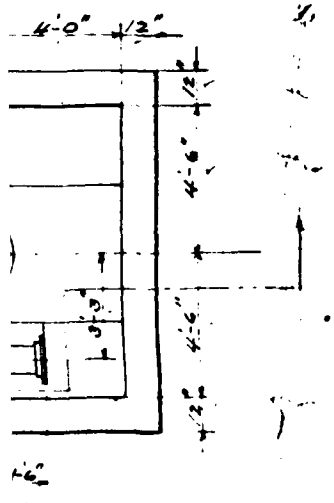
TIVOLI LAKE DAM.

DETAILS OF SLUICE WAY AND BRIDGE


7'-6"
15'-0"
7'-6"
5'-0"
8'-0"
7'-6"
15'-0"
7'-6"


6'-6"
4'-3"
30'-0"
45'-3"
7'-8"
4'-0"
C.LINE OF SLUCE.
10" STEEL PIPE
GATE VALVE



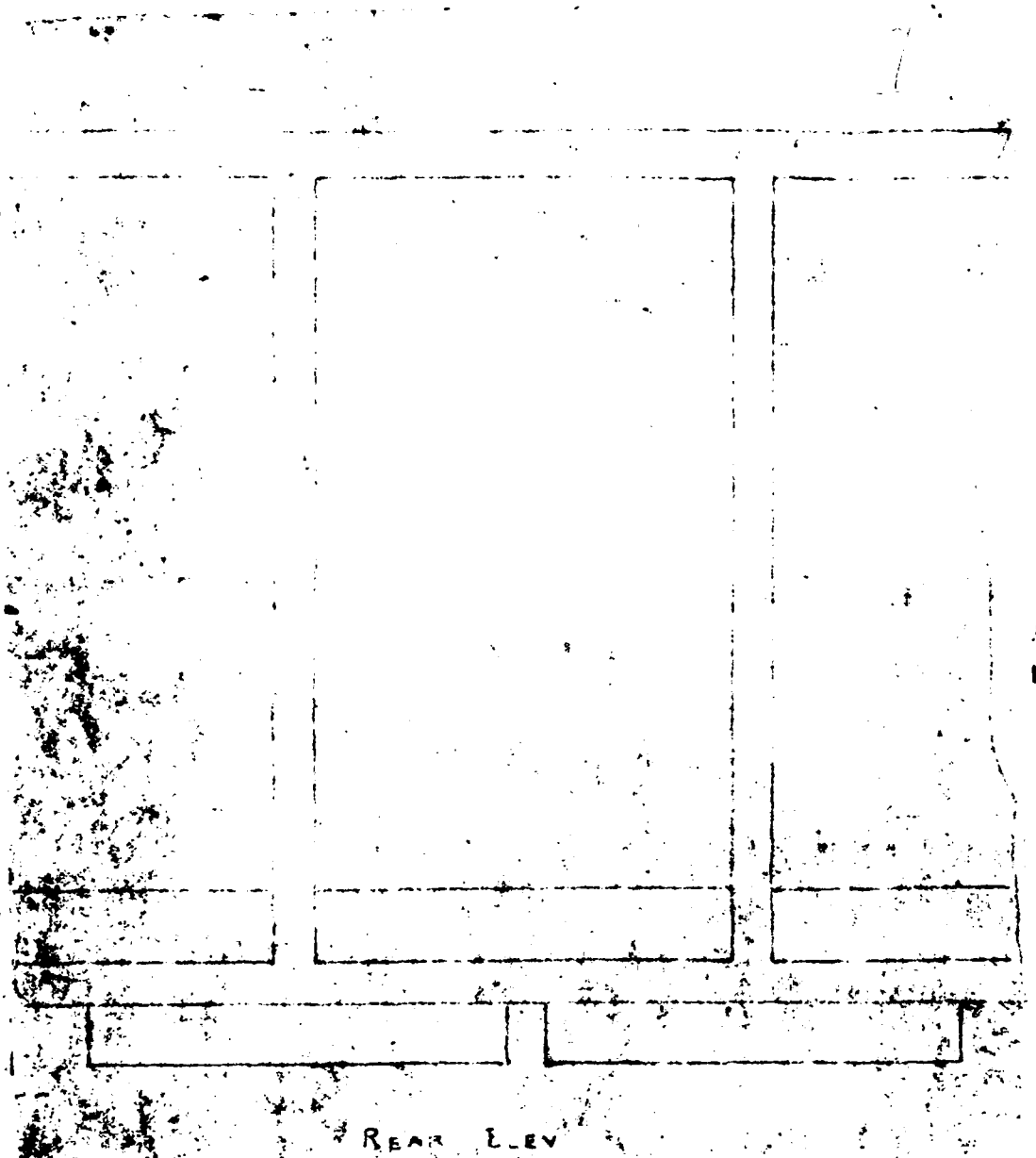


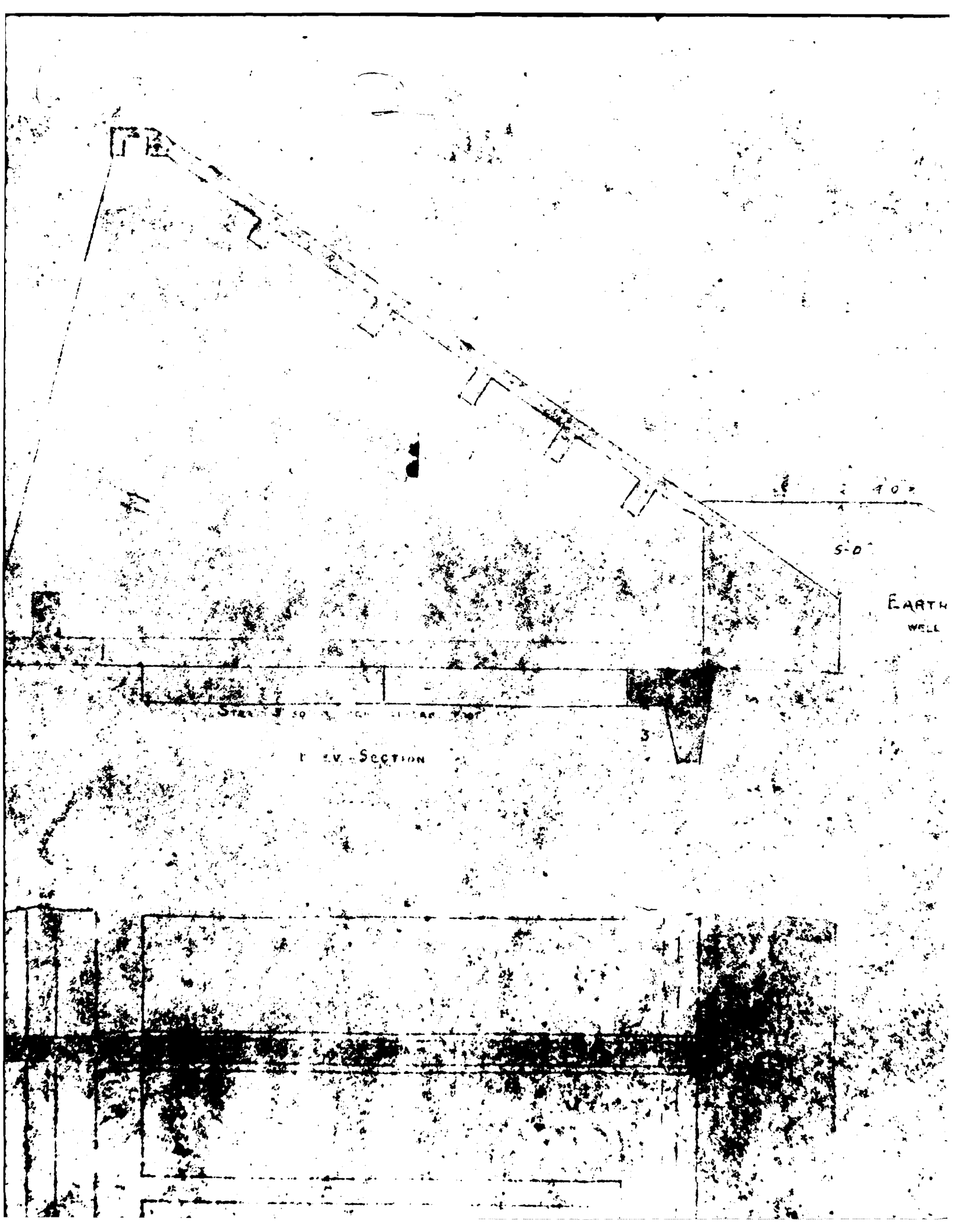
TIVOLI LAKE DAM.

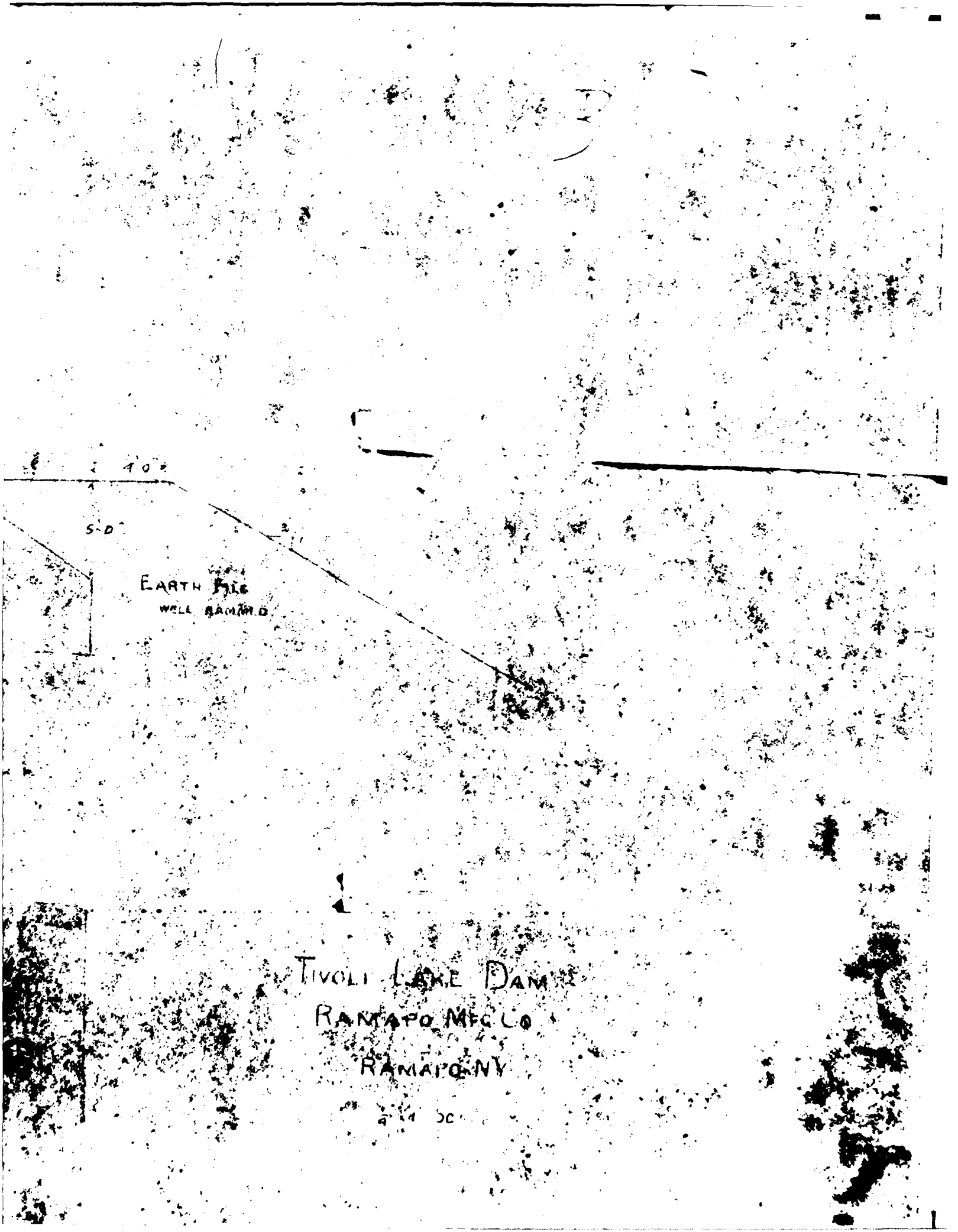
DETAILS OF SLUCE WAY AND BRIDGE

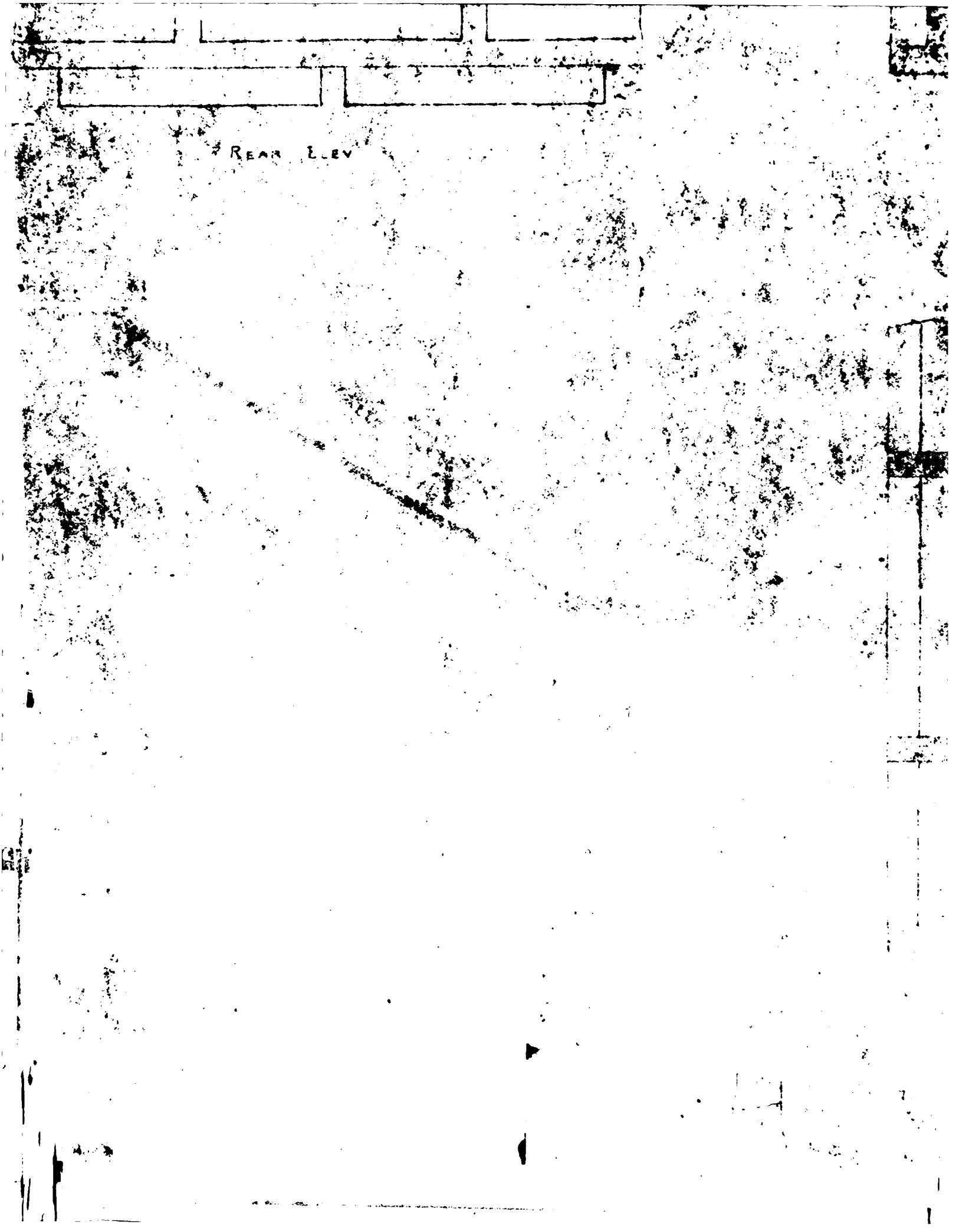
CLARK & COMPANY 100 BROADWAY, N.Y.

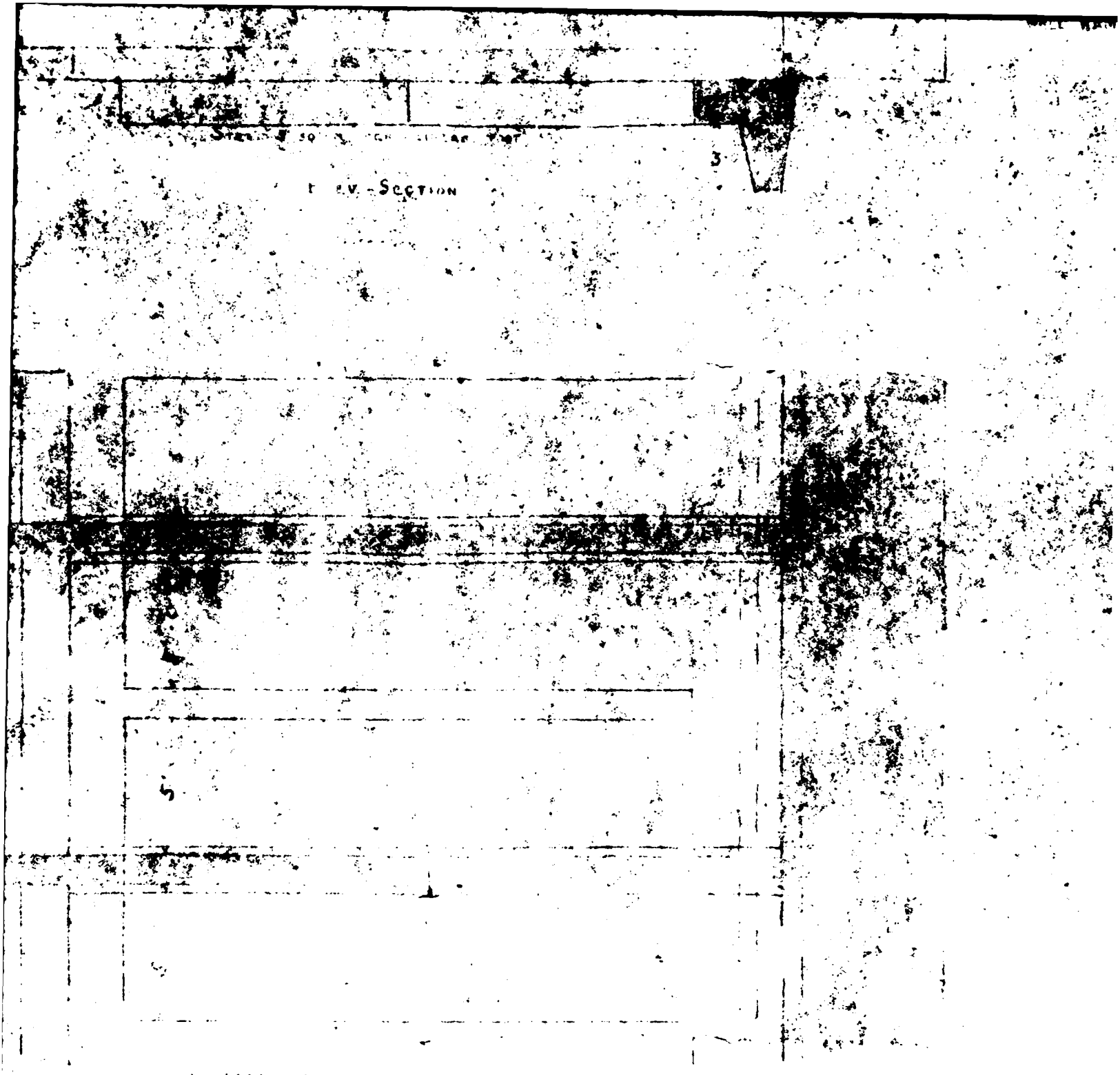
SCALE 1"=1'-0"


REAR ELEV


5-0
EARTH
WELL


1 : 1.0
5·0
EARTH FILL
WELL RAMMED
TIVOLI LAKE DAM
RAMAPO MFG CO
RAMAPO NY

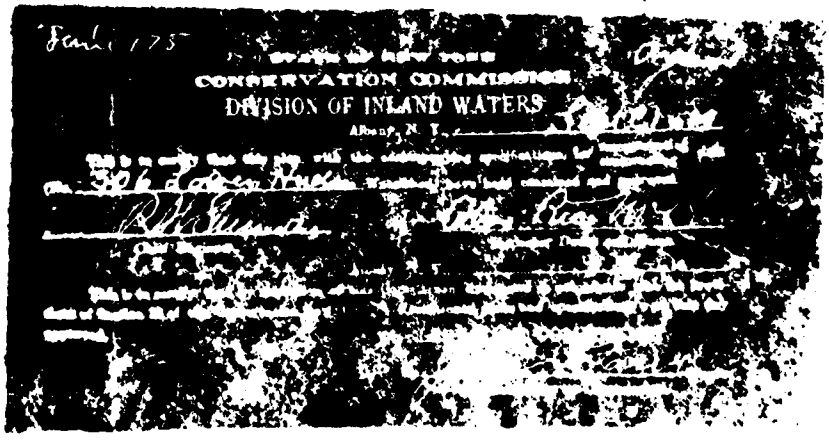

REAR ELEV

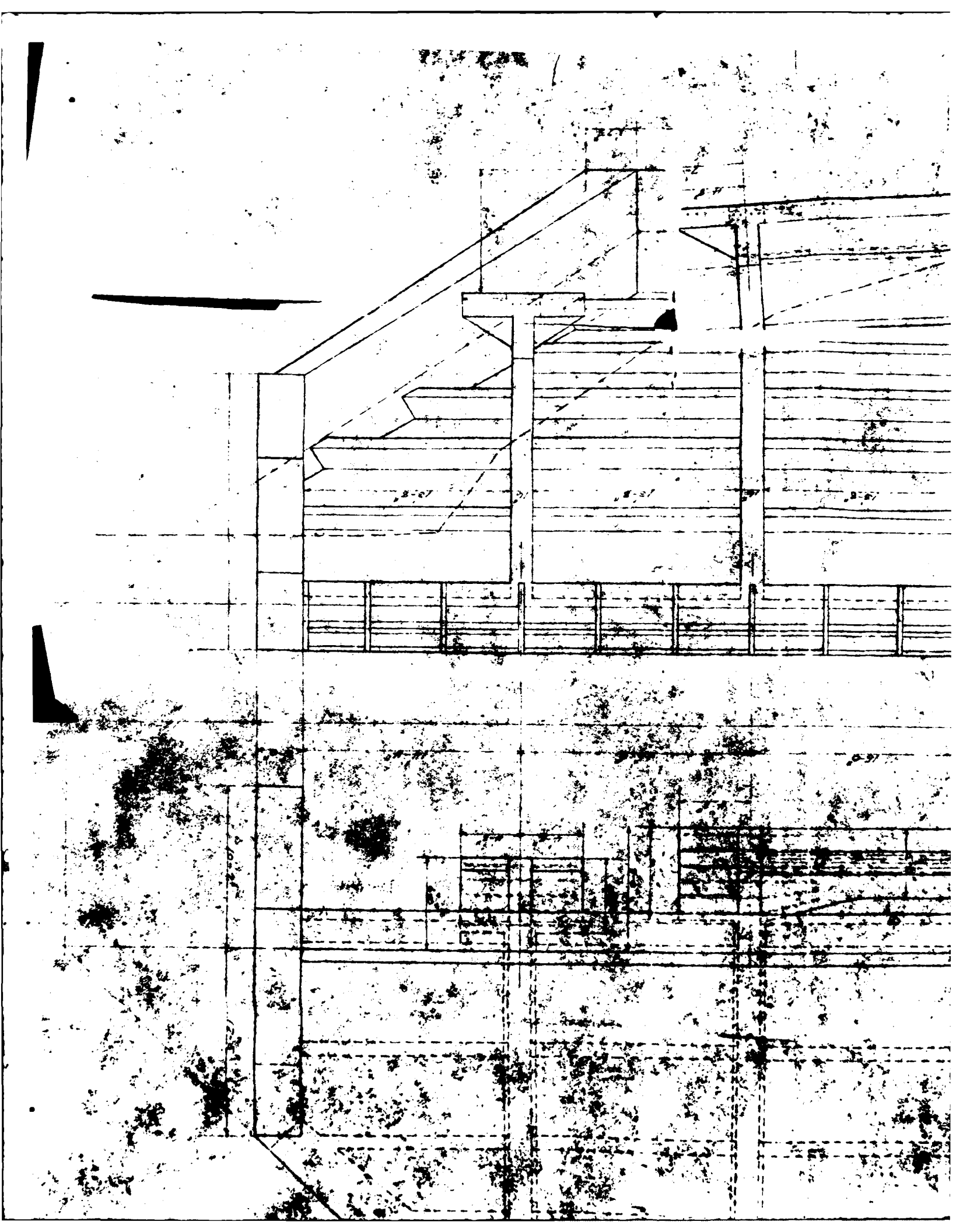

3
REV. - SECTION

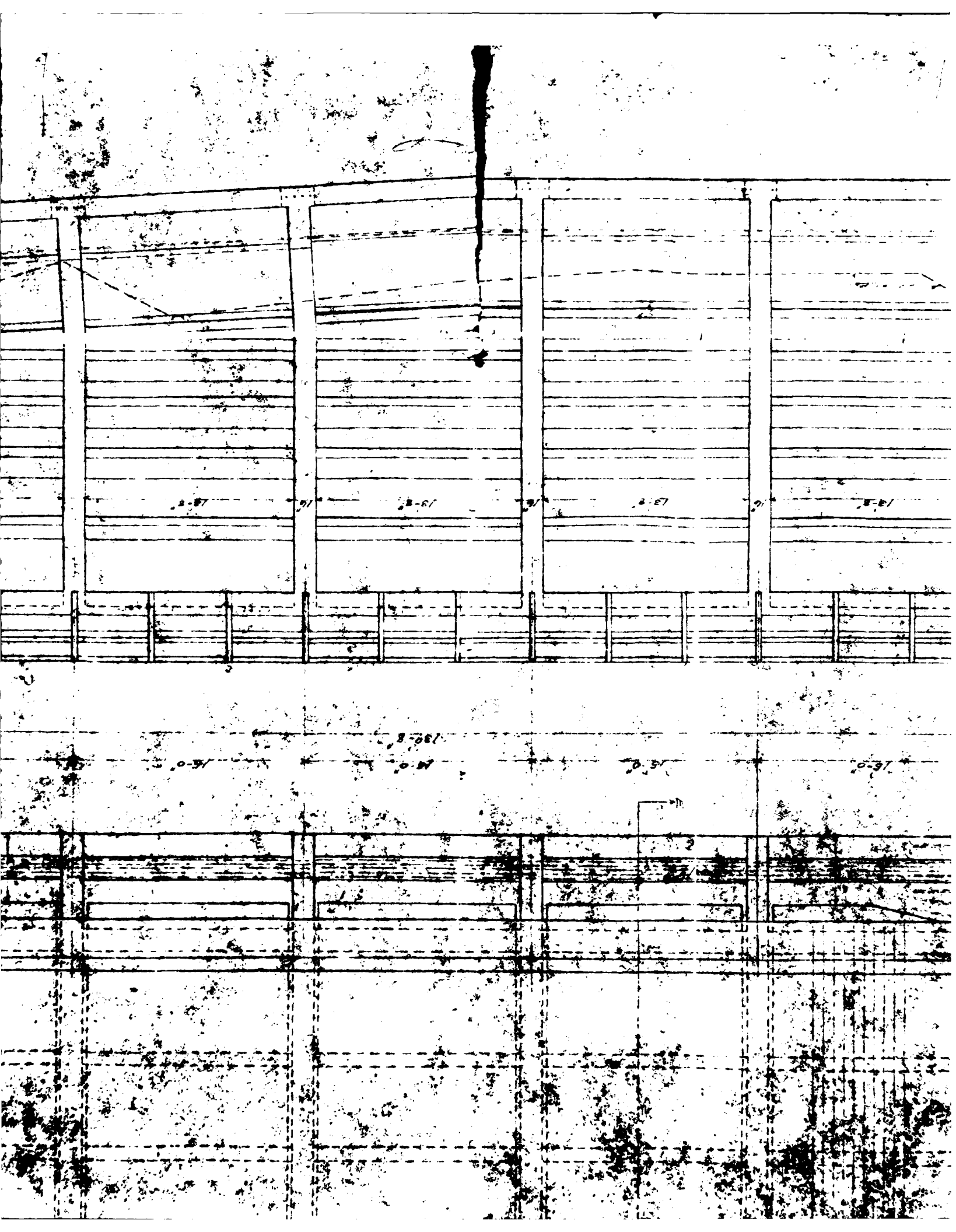
TIVOLI LAKE DAM

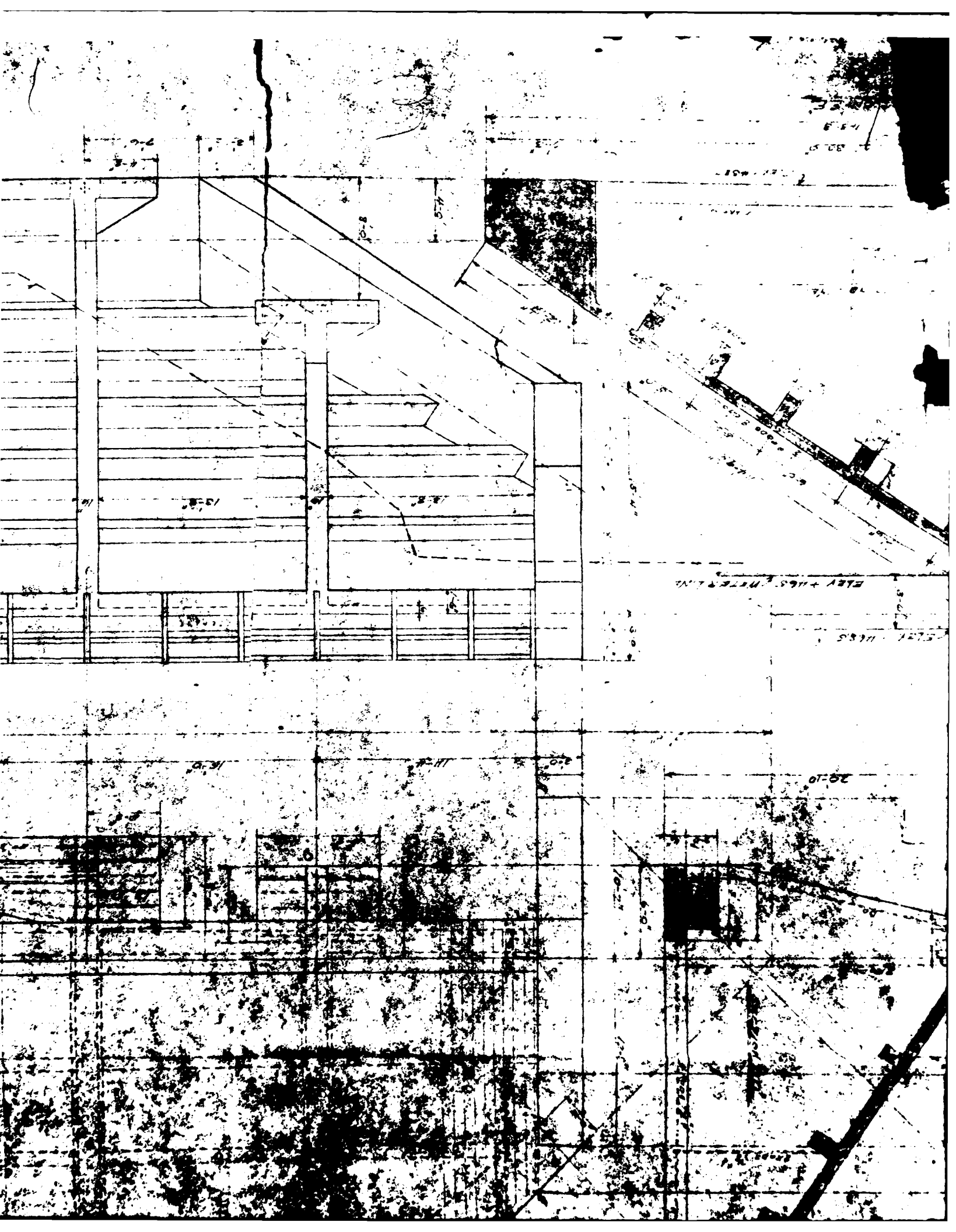
RAMAPO MFG CO

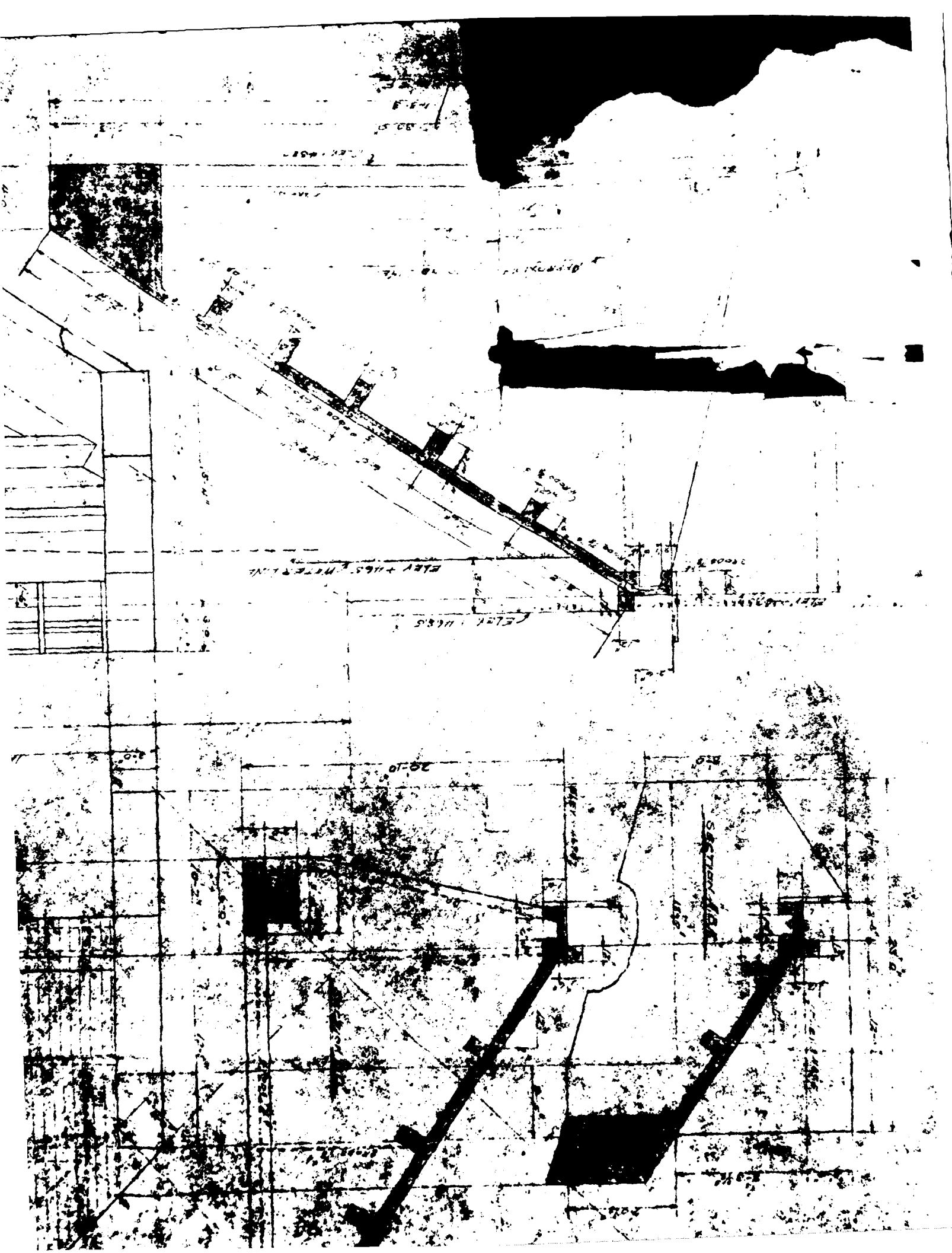
RAMAPO NY

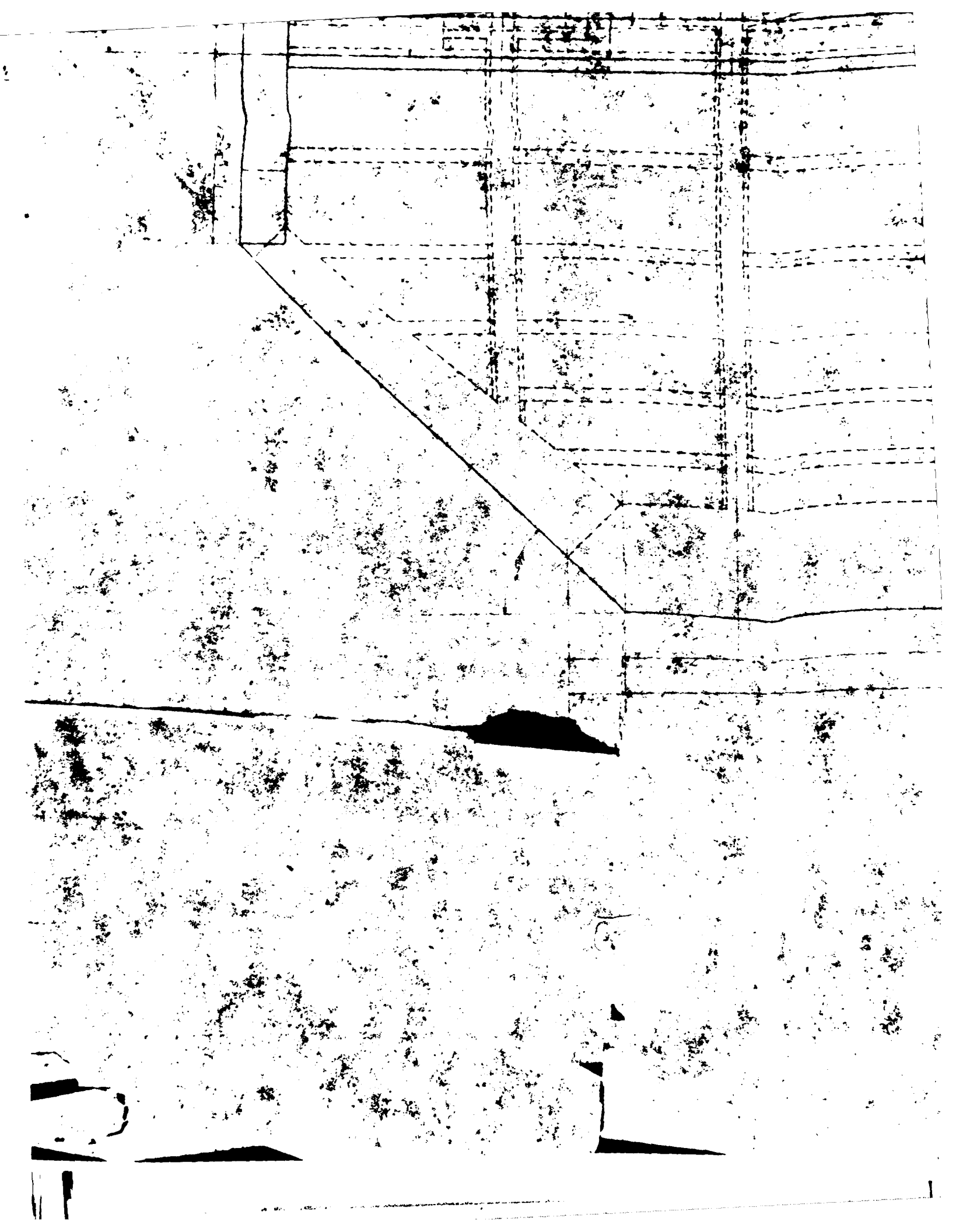












REINFORCED CONCRETE DAM

FOR THE

PATHAT WATER CO.

AT

TIVOLI LAKE

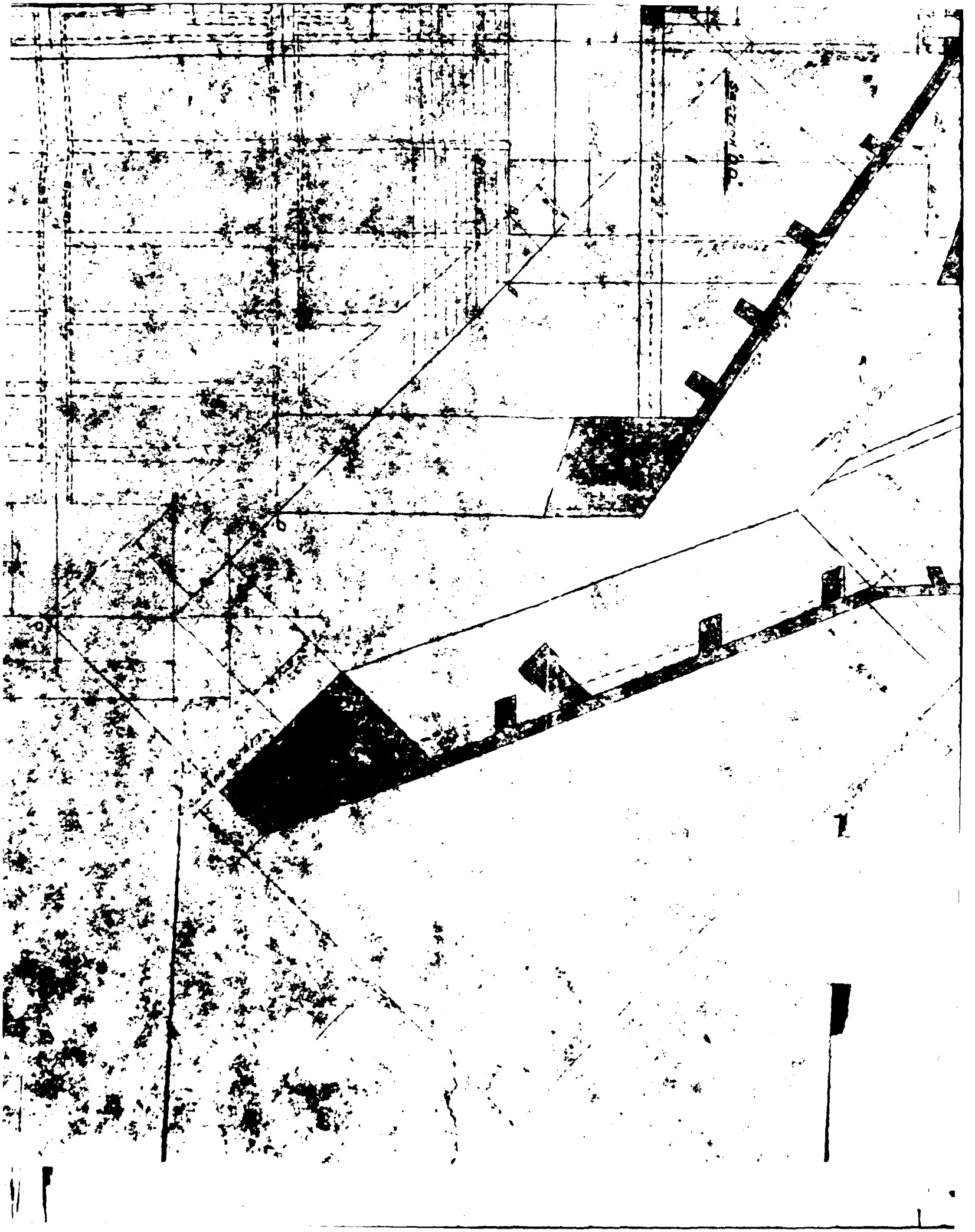
NEAR SPARKILL ROCKLAND CO. NEW YORK

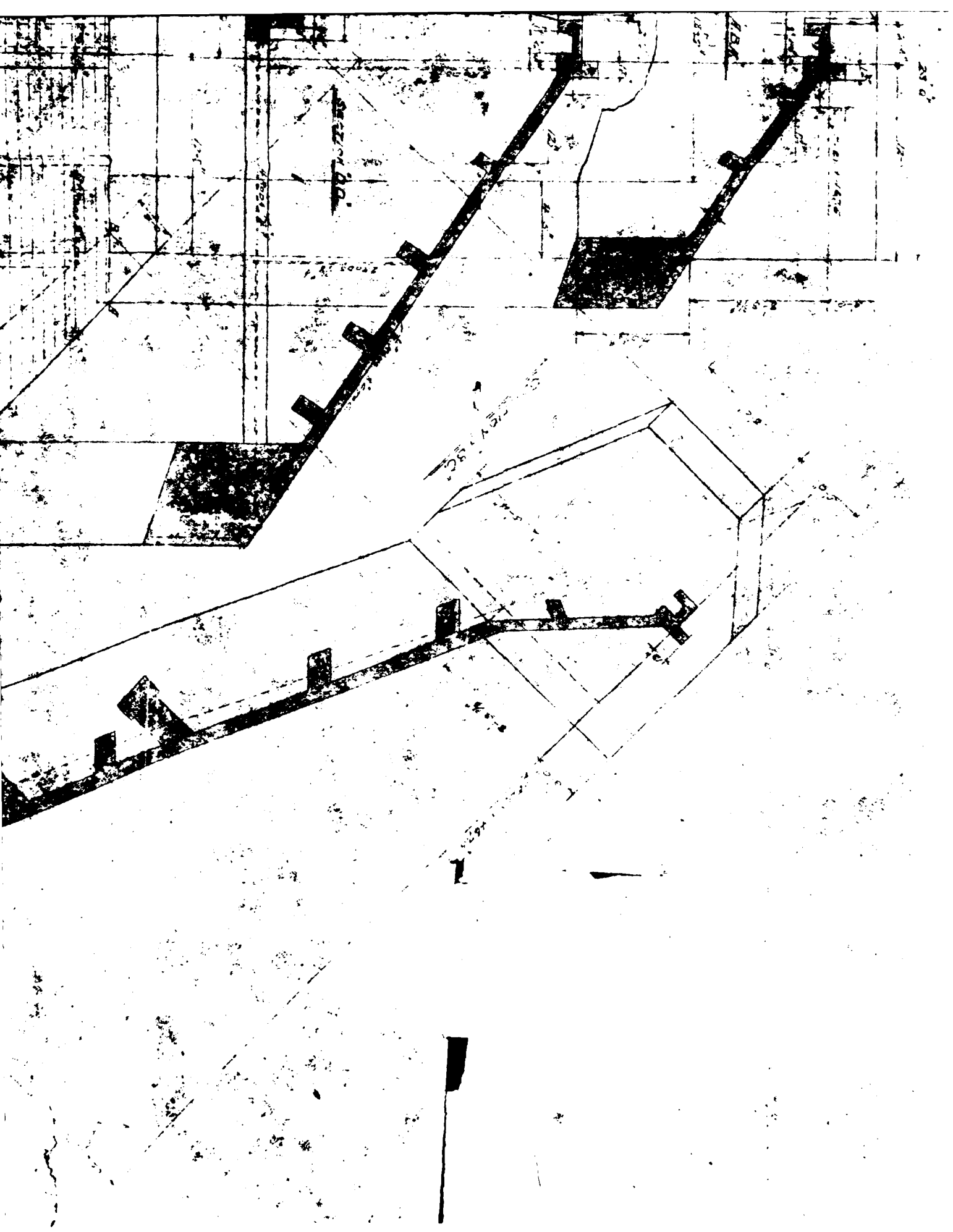
CLARK & COLLEY CONSTRUCTION CO.

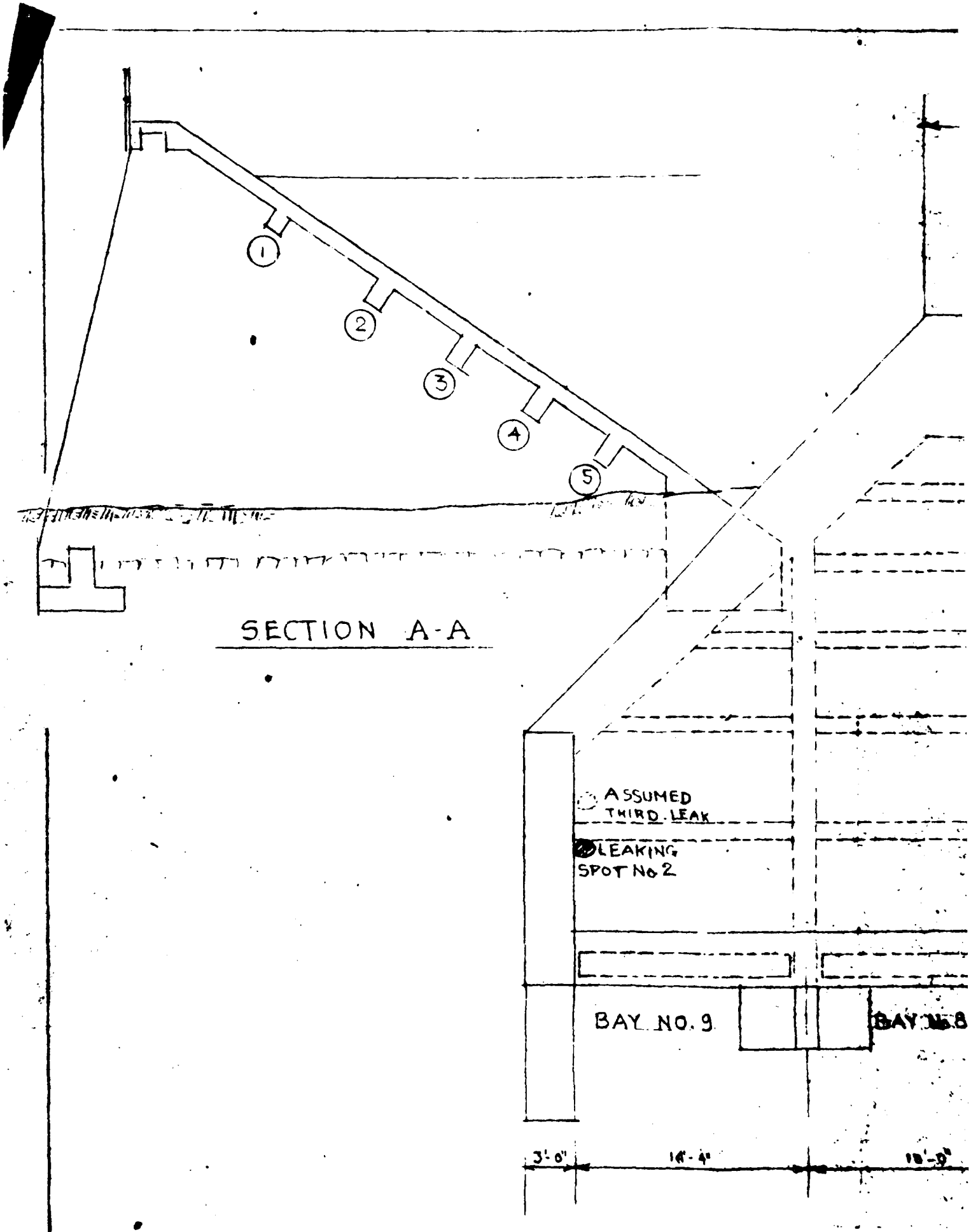
120 BROADWAY N.Y. CITY

SCALE 1"=5'-0"

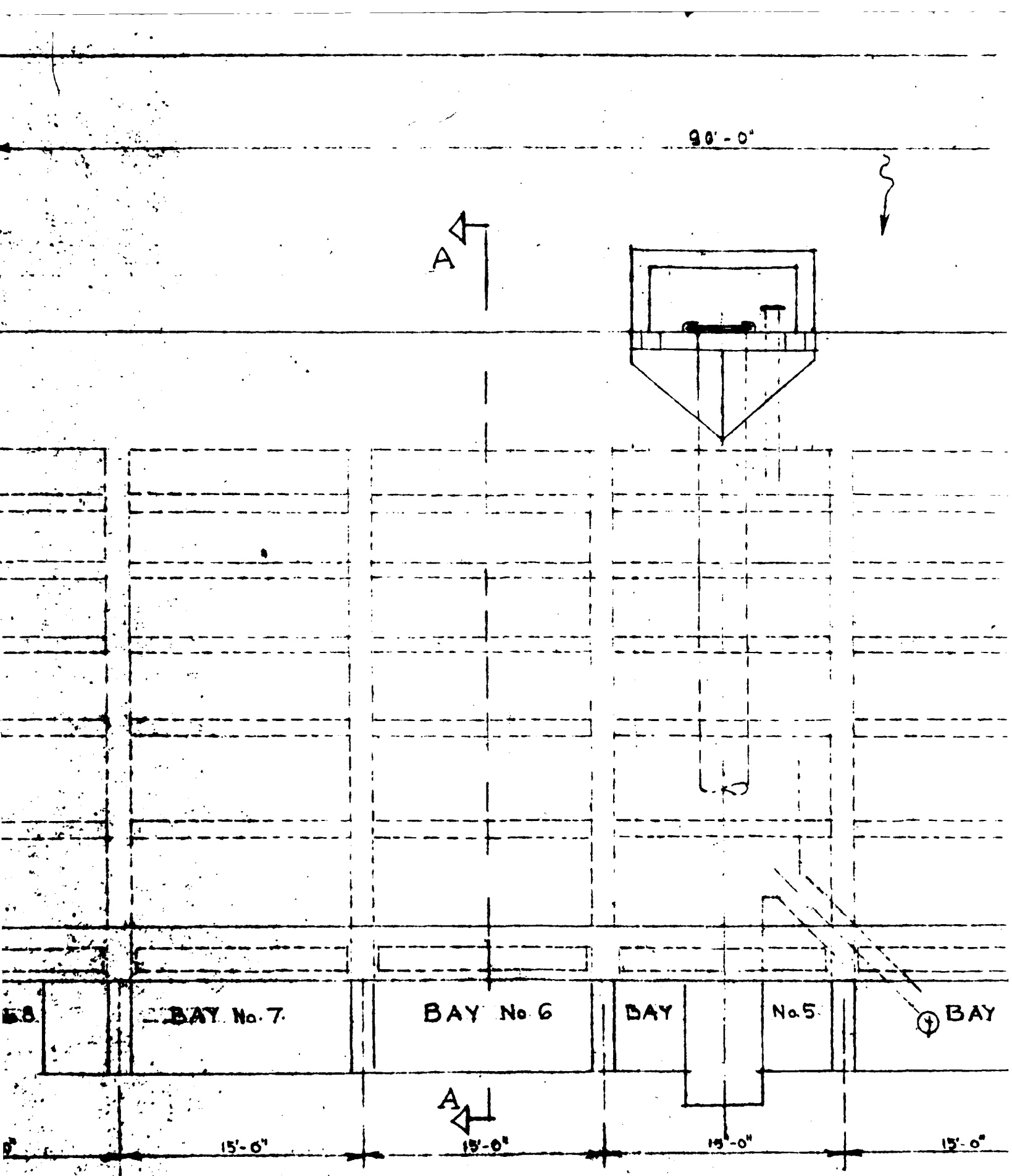
90'-0"






1
2
3
4
5
ASSUMED THIRD LEAK
2 LEAKING SPOT No 2
BAY NO. 9
BAY No. 8
3'-0"
14'-4"
13'-9"

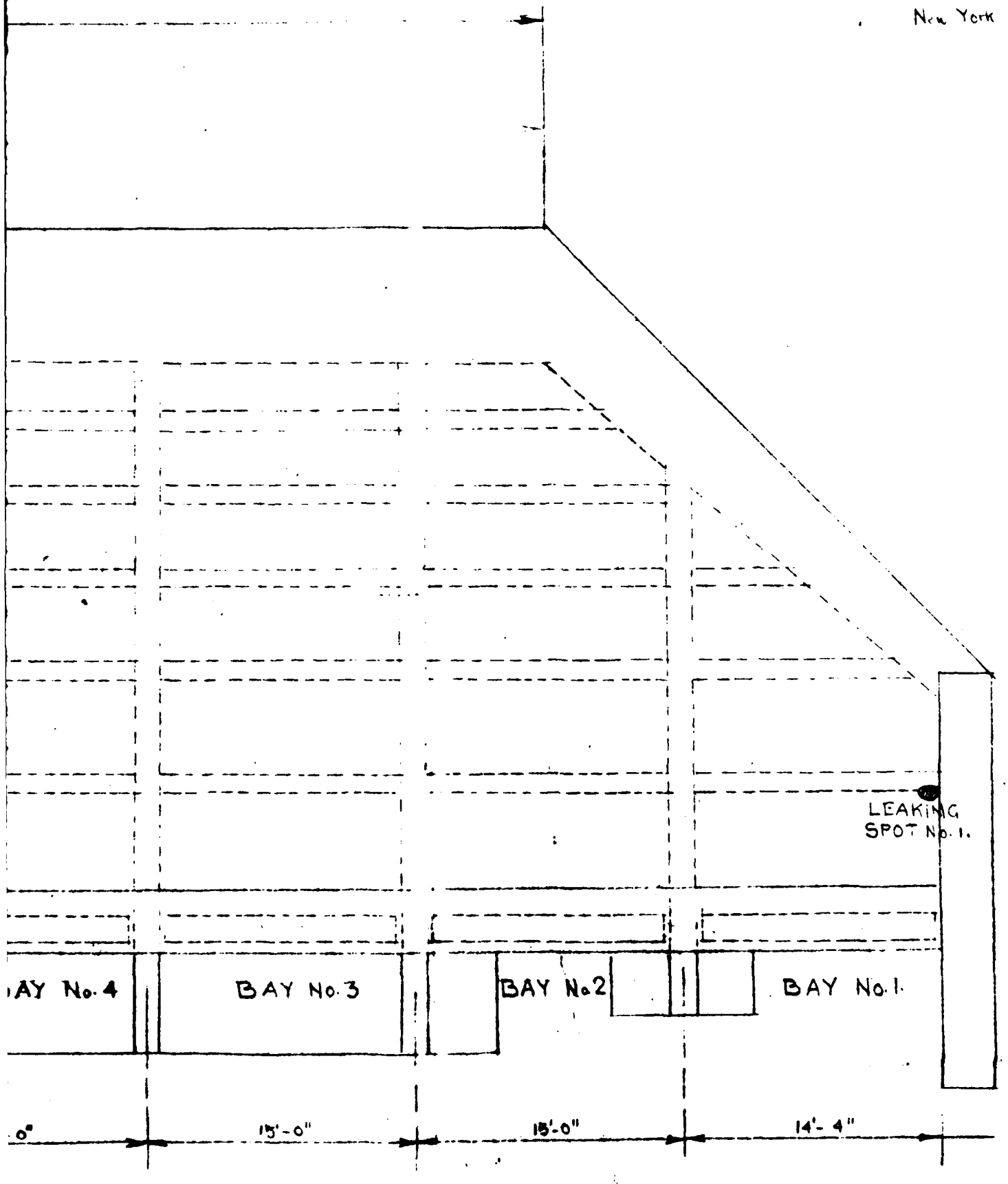
SECTION A-A

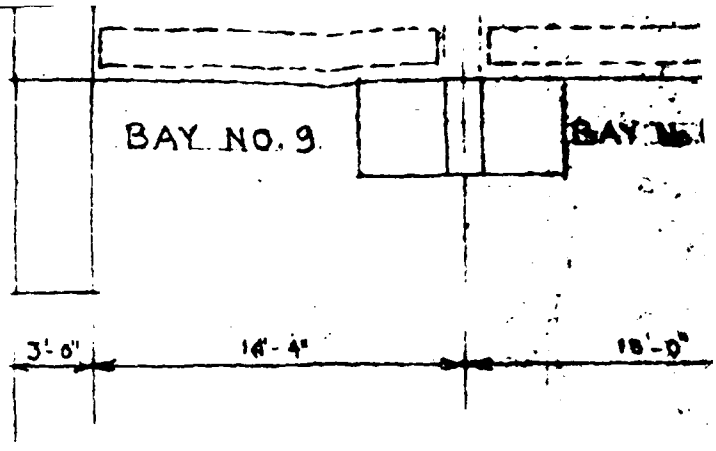


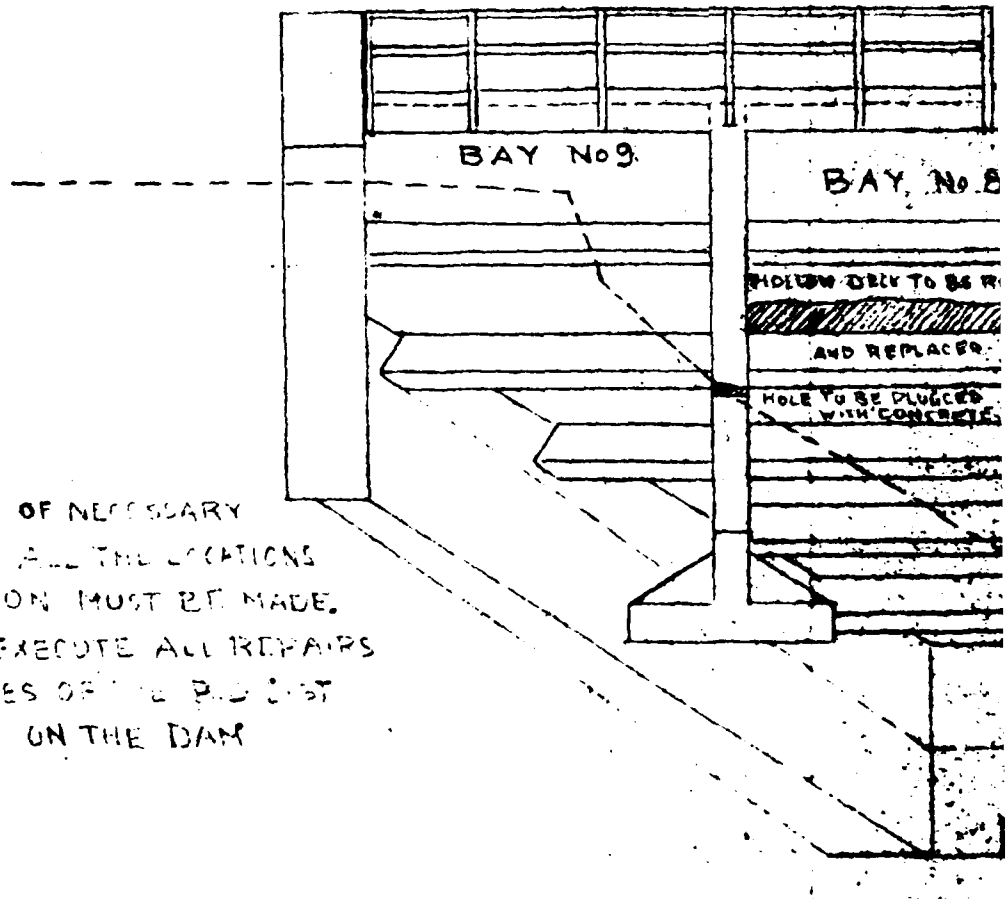
PLAN OF CRANBERRY DAM
(FORMER TIVOLI DAM)

LOCATION OF NECESSARY CONCRETE REPAIRS

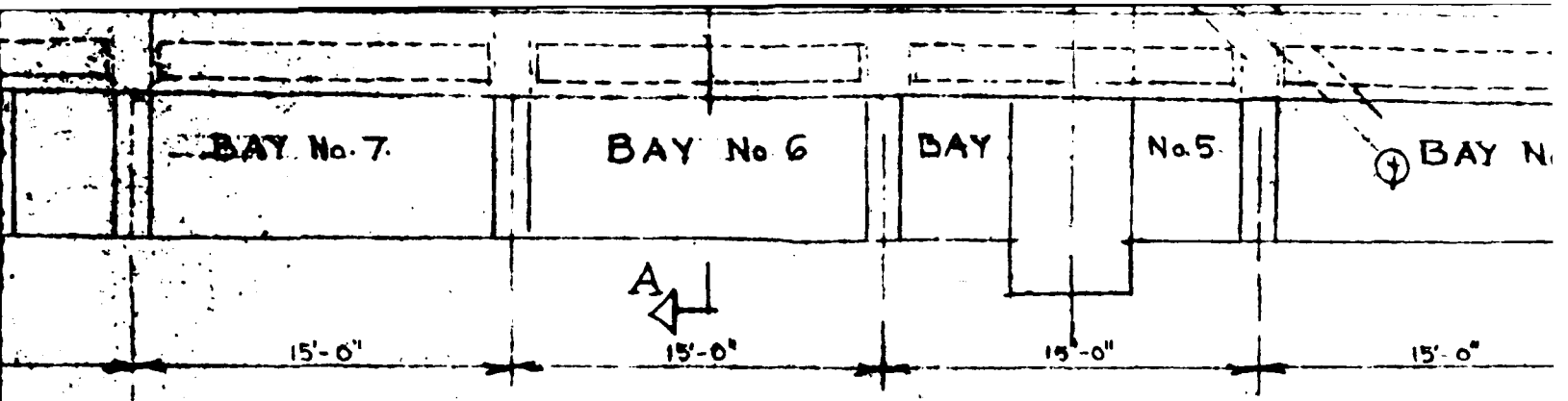
POTHAT WATER COMPANY
SLOATSBURG
New York





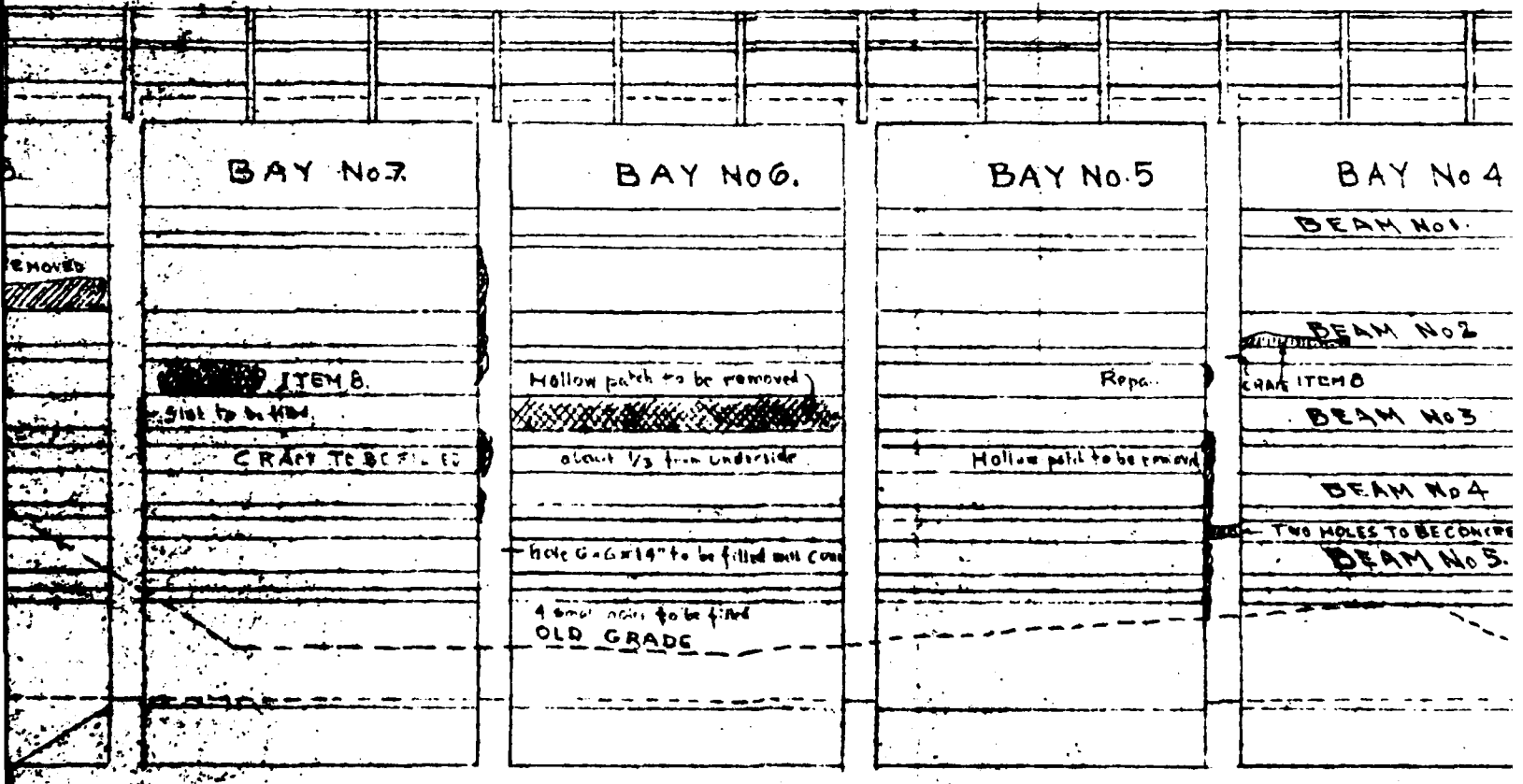


NOTE:
THE INDICATED SPOTS OF NECESSARY REPAIRS DO NOT SHOW ALL THE LOCATIONS WHERE REHABILITATION MUST BE MADE. CONTRACTOR SHALL EXECUTE ALL REPAIRS WITHIN THE QUANTITIES OF THE PROJECT WHICH HE WILL FIND ON THE DAM

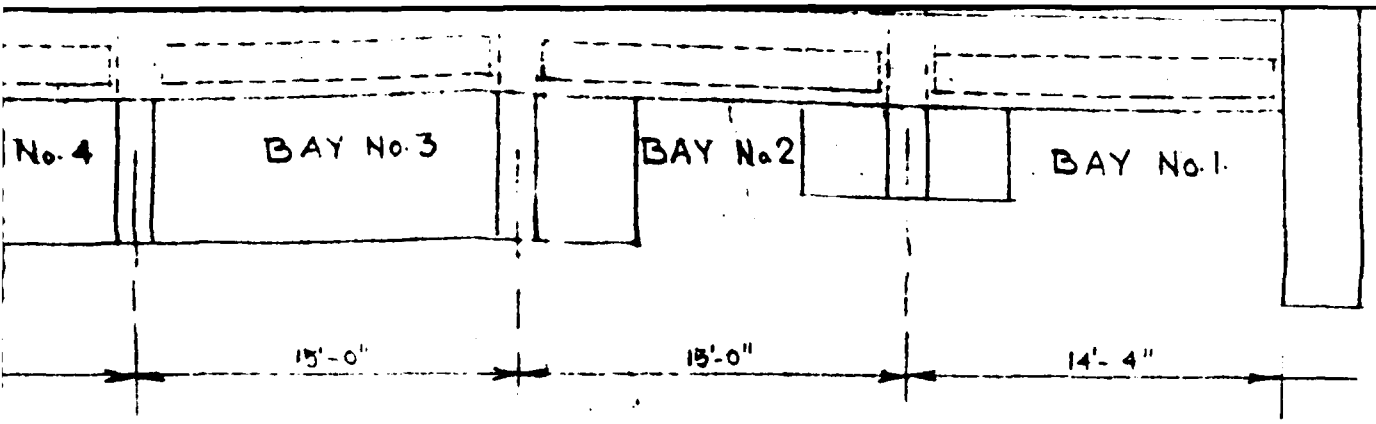


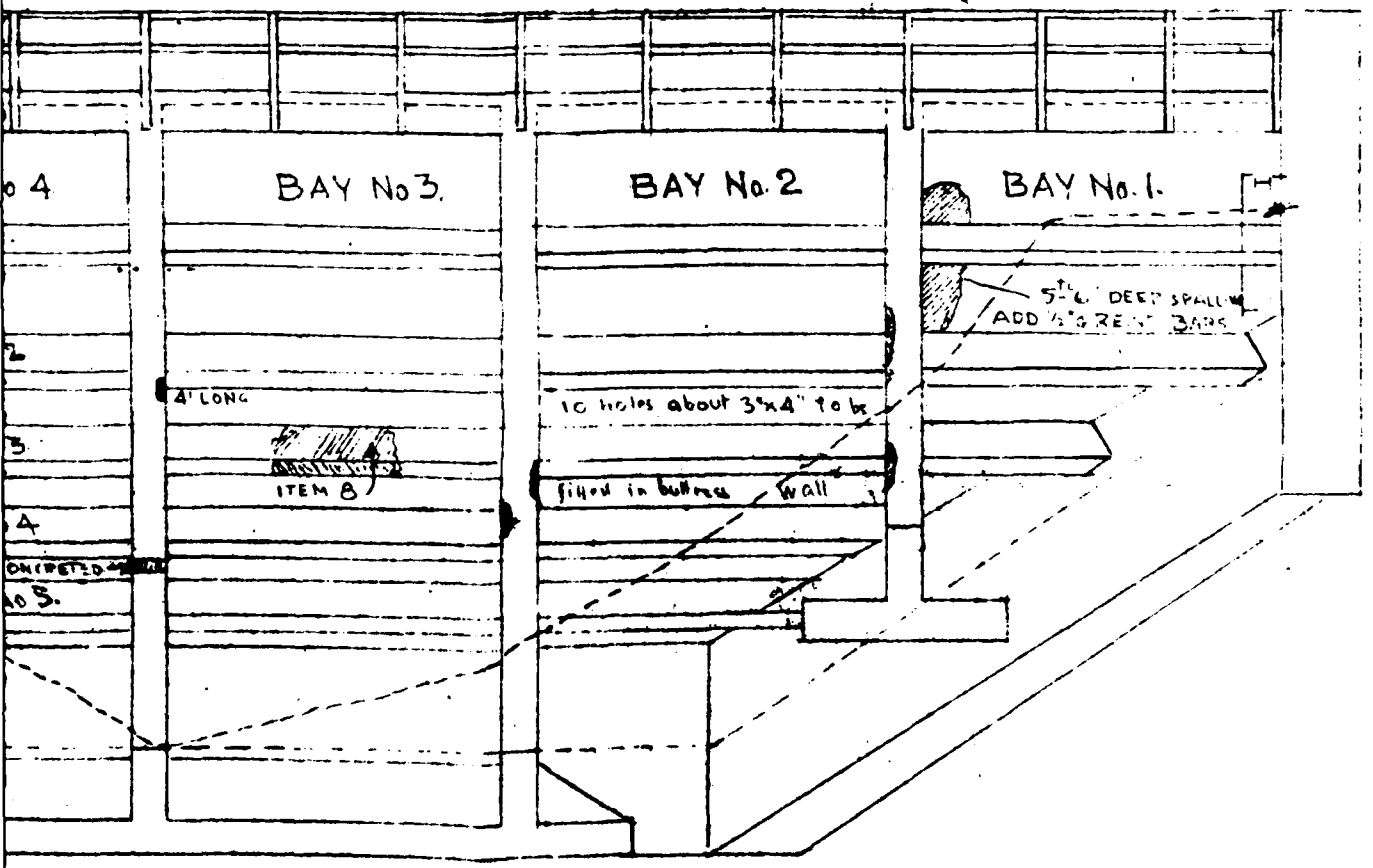
PLAN OF CRANBERRY DAM
(FORMER TIVOLI DAM)

LOCATION OF NECESSARY CONCRETE REPAIRS.



ELEVATION OF DAM LOOKING UPSTREAM.





ITEM 5

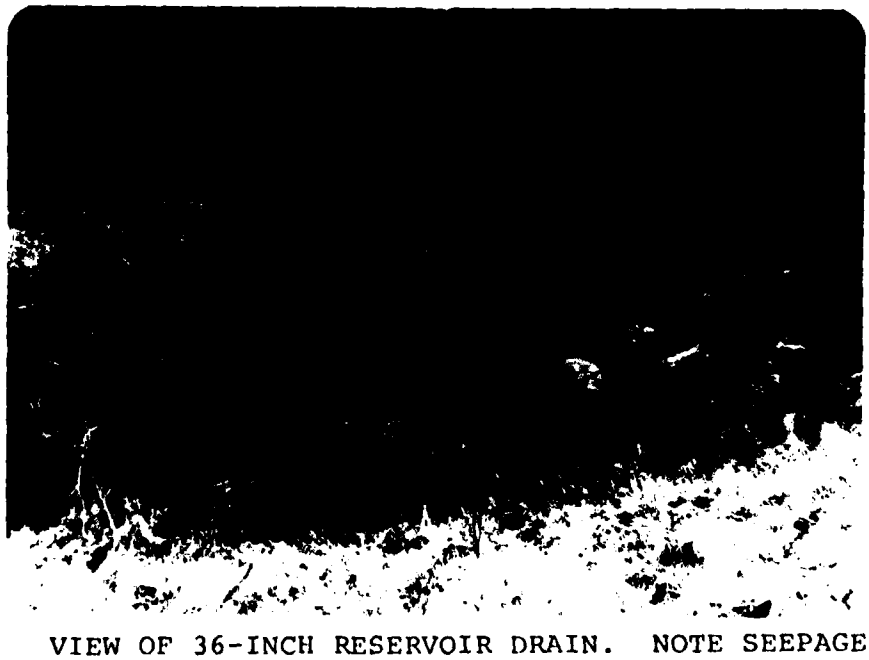
NEW YORK, 8. AUGUST 1974

NEW YORK STATE PROFESSIONAL ENGINEER'S LICENSE No. 28112.

PHOTOGRAPHS

APPENDIX B

2. VIEW OF UPSTREAM FACE AND CONTROL FOR 36-INCH RESERVOIR DRAIN. NOTE ACCESS BRIDGE NONEXISTANT.



3. VIEW OF 36-INCH RESERVOIR DRAIN. NOTE SEEPAGE OUT OF PIPE.

4. VIEW OF CONTROL FOR 10-INCH RESERVOIR DRAIN AT TOE OF DAM.

5. VIEW OF OUTLET FOR 10-INCH RESERVOIR DRAIN.

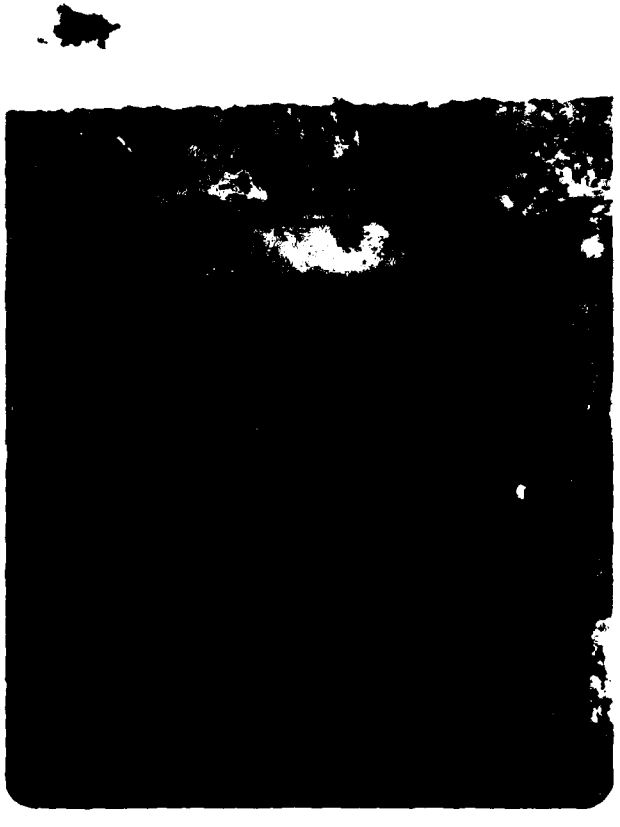
6. VIEW OF SPILLWAY CREST

7. VIEW OF CAUSWAY ACROSS LAKE NEAR SPILLWAY.

8. VIEW OF SEEPAGE THROUGH BUTTRESS - BAYS 8 AND 9. NOTE REPAIRS TO DECK AND BEAM.

9. VIEW OF SEEPAGE THROUGH RIGHT ABUTMENT. NOTE SLOPE CONDITION AND EVIDENCE OF FRENCH DRAINS.

10. VIEW OF SEEPAGE THROUGH BUTTRESS IN BAY 2.



11. VIEW OF SEEPAGE THROUGH FACE OF DAM IN BAY 5. NOTE PATCHES IN FACE.

VISUAL INSPECTION CHECKLIST

APPENDIX C

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

Name of Dam TIVOLI LAKE (Cranberry)

Fed. I.D. # NY 52 DEC Dam No. 306 AND 309 (spillway)

River Basin Lower Hudson

Location: Town Ramapo County Rockland

Stream Name Ramapo River

Tributary of Hudson River

Latitude (N) 41° 8' Longitude (W) 74° 12'

Type of Dam Concrete - Ambersen

Hazard Category HIGH * Few structures immediately adjacent but Sloatsburg could be damaged

Date(s) of Inspection April 24, 1980

Weather Conditions Fair - 60-65°

Reservoir Level at Time of Inspection Full (WSL. = 509.1)

b. Inspection Personnel Kalman Szalay - Principal Geotechnical Engr.

Joseph Fiteni Jr. Geotechnical Engineer

c. Persons Contacted (Including Address & Phone No.) H. Pierson Mapes -

Ramapo Land Company, P.O. Box 415 Sloatsburg N.Y. 10974 - 753-8228 (914)

Mr. Scott Vanderhoes - same as above

Mr. Anthony Spadavecchia - " " "

d. History:

Date Constructed 1908 Date(s) Reconstructed N.A.

Designer Clark and Company - New York, N.Y.

Constructed By As Above

Owner Pothat Water Company - Ramapo Land Company

2) Embankment NONE

a. Characteristics

(1) Embankment Material _____

(2) Cutoff Type _____

(3) Impervious Core _____

(4) Internal Drainage System _____

(5) Miscellaneous _____

b. Crest

(1) Vertical Alignment _____

(2) Horizontal Alignment _____

(3) Surface Cracks _____

(4) Miscellaneous _____

c. Upstream Slope

(1) Slope (Estimate) (V:H) _____

(2) Undesirable Growth or Debris, Animal Burrows _____

(3) Sloughing, Subsidence or Depressions _____

(4) Slope Protection _____

_____

_____

(5) Surface Cracks or Movement at Toe _____

_____

d. Downstream Slope

(1) Slope (Estimate - V:H) _____

(2) Undesirable Growth or Debris, Animal Burrows _____

_____

(3) Sloughing, Subsidence or Depressions _____

_____

_____

(4) Surface Cracks or Movement at Toe _____

_____

(5) Seepage _____

_____

_____

(6) External Drainage System (Ditches, Trenches; Blanket) _____

_____

_____

(7) Condition Around Outlet Structure _____

_____

(8) Seepage Beyond Toe _____

_____

e. Abutments - Embankment Contact

_____

_____

(1) Erosion at Contact _____

(2) Seepage Along Contact _____

3) Drainage System

a. Description of System None

b. Condition of System _____

c. Discharge from Drainage System _____

4) Instrumentation (Monumentation/Surveys, Observation Wells, Weirs, Piezometers, Etc.) None

5) Reservoir

a. Slopes Rock outcrop and stable glacial till

b. Sedimentation - none evident

c. Unusual Conditions Which Affect Dam None

6) Area Downstream of Dam

a. Downstream Hazard (No. of Homes, Highways, etc.) None immediately downstream - however village of Slatsburg about 1 mi DS.

b. Seepage, Unusual Growth Considerable Seepage Downstream and on right abutment. French drain installed D/S to collect.

c. Evidence of Movement Beyond Toe of Dam None

d. Condition of Downstream Channel Some brush growth - generally well formed stream channel.

7) Spillway(s) (Including Discharge Conveyance Channel)

Simple overflow monolith located Northeast of Dam about 1 mile. only one spillway

a. General Spillway is simple gravity overflow monolith about 5Ft high (165Ft long) Some concrete recently repaired. generally good condition - spillway flowing in certain sections during inspection.

b. Condition of Service Spillway generally good - concrete repairs carried out in 1974. Some movement evident, and some concrete spalling at joints. - Siltation evident upstream.

c. Condition of Auxiliary Spillway NA

d. Condition of Discharge Conveyance Channel - Free flowing, although filled with boulders and tree debris

8) Reservoir Drain/Outlet

Type: Pipe Castiron 10" Conduit 36" Conc. Other

Material: Concrete 36" Metal 10" Other

Size: 1 @ 36" 1 @ 10" Length - 147.2± ft

Invert Elevations: Entrance — Exit —

Physical Condition (Describe): Unobservable 10 inch is unobservable

Material: 10" cast iron is unobservable (buried for entire length)

Joints: Some seepage and spalling Alignment Some vertical deflection (low

Structural Integrity: generally good - local spalling

Hydraulic Capability: 10" —

36" Sluice - None - gate inoperable

Means of Control: Gate X(36") Valve 10" Uncontrolled

Operation: Operable 10" Inoperable 36" X Other

Present Condition (Describe): 10" appears to be in good condition 36" located on inaccessible structure - connection of control to gate non existent - good seal on control gate to 36" pipe only minor leakage.

9) Structural

a. Concrete Surfaces generally good - repaired at various times - some seepage and spalling at joints on up and downstream faces

b. Structural Cracking - some on buttresses - bay 3 - 9.

c. Movement - Horizontal & Vertical Alignment (Settlement) None visible

d. Junctions with Abutments or Embankments Recently repaired - Right - Wing Wall good condition - Left - Riprapped - seepage evident -

e. Drains - Foundation, Joint, Face None evident - French Drains in foundation downstream to collect seepage -

f. Water Passages, Conduits, Sluices 36 inch concrete sluice, 10 inch outlet pipe

g. Seepage or Leakage seepage through concrete decking in bays 2, 5, 8, 9. through buttress at bay 8 and 9.

h. Joints - Construction, etc. Some seepage, natural above, appears to be located at construction joints

i. Foundation hardpan - some seepage through foundation is evident.

j. Abutments left appears stable
right exhibits high seepage at ground p.

k. Control Gates 36" sluice gate inoperable

l. Approach & Outlet Channels None

m. Energy Dissipators (Plunge Pool, etc.) None

n. Intake Structures None

o. Stability Appears Stable

p. Miscellaneous

HYDROLOGIC DATA AND COMPUTATIONS

APPENDIX D

MAY 22, 1980

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**********
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
 LAST MODIFICATION  26 FEB 79
**********
     1          A1       PHASE 1 INSPECTION TIVOLI DAM ROCKLAND COUNTY                    1551-08
     2          A2            TEST OF SPILLWAY ADEQUACY
     3          A3                    TAMS ENGINEERS AND ARCHITECTS     MAY 1980
     4          B    100       0      30       0       0       0       0       0       0       0
     5          B1     5
     6          J      1       2       1
     7          J1     1     0.5
     8          K              1
     9          K1        INFLOW HYDROGRAPH COMPUTATION FOR TIVOLI LAKE
    10          M      1      -1    2.98                                                    1
    11          P             24.5   106.1   122.4   134.7     151
    12          T                                                       1     0.1             0.055
    13          U     16
    14          U1   300     840     835     570     390     275     180     130      90      65
    15          V1    40      30      15      10       5       0
    16          X     -1    -.05       3
    17          K      1       2
    18          K1        ROUTE HYDROGRAPH THROUGH RESERVOIR
    19          Y                               1       1
    20          Y1     1                                                       790      -1
    21          Y4   509     511   512.5     513     516
    22          Y5     0    1470    3350    4225   13930
    23          $S     0     790    1004    1170    1228    1580
    24          $E   485     509     511   512.5     513     514
    25          $$ 509.
    26          $D 512.5       0
    27          K      1       3
    28          K1        CHANNEL ROUTING   STN 7+00
    29          Y                               1       1
    30          Y1     1
    31          Y4  0.04    0.04    0.04     440     520     700    0.05
    32          Y7  3650     520    3690     500    3880     480    3960     460    4000     440
    33          Y7  4040     460    4075     480    4100     500    4300     520
    34          K      1       4
    35          K1        CHANNEL ROUTING STN 16+00
    36          Y                               1       1
    37          Y1     1
    38          Y4  0.04    0.04    0.04     385     440     900   0.061
    39          Y7  5700     440    5820     420    5880     400    6000     385    6390     396
    40          Y7  6430     397    6490     400    6590     420    6760     440
    41          K      1       5
    42          K1        CHANNEL ROUTING STN 26+00
    43          Y                               1       1
    44          Y1     1
    45          Y4  0.04    0.04    0.04     359     420    1000   0.026
    46          Y7  4700     420    4790     400    5700     380    6000     359    6180     365
    47          Y7  6370     360    6410     361    6510     380    6580     400    6650     420
    48          K     99
```

PREVIEW OF SEQUENCE OF STREAM NETWORK CALCULATIONS

RUNOFF HYDROGRAPH AT 1
ROUTE HYDROGRAPH TO 2
ROUTE HYDROGRAPH TO 3
ROUTE HYDROGRAPH TO 4
ROUTE HYDROGRAPH TO 5
END OF NETWORK

****
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION JULY 1978
LAST MODIFICATION 26 FEB 79
****

RUN DATE= 80/05/22.
TIME= 15.07.18.

PHASE 1 INSPECTION TIVOLI DAM ROCKLAND COUNTY 1551-08
IESI OF SPILLWAY ADEQUACY
TAMS ENGINEERS AND ARCHITECTS MAY 1980

JOB SPECIFICATION

NQ	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
100	0	30	0	0	0	0	0	0	0

JOPER	NWT	LROPT	TRACE
5	0	0	0

MULTI-PLAN ANALYSES TO BE PERFORMED
NPLAN= 1 NRTIO= 2 LRTIO= 1

RTIOS= 1.00 .50

****

SUB-AREA RUNOFF COMPUTATION

INFLOW HYDROGRAPH COMPUTATION FOR TIVOLI LAKE

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
1	0	0	0	0	0	0	0	0

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	-1	2.98	0.00	2.98	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	24.50	106.10	122.40	134.70	151.00	0.00	0.00

TRSPC COMPUTED BY THE PROGRAM IS .800

LOSS DATA

LROPT	STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0	0.00	0.00	1.00	0.00	0.00	1.00	1.00	.10	0.00	.06

GIVEN UNIT GRAPH, NUMGO= 16

300.	880.	835.	570.	390.	275.	180.	130.	90.	65.
40.	30.	15.	10.	6.	0.				

UNIT GRAPH TOTALS 3815. CFS OR .99 INCHES OVER THE AREA

RECESSION DATA
STRTQ= -1.00 QRCSN= -.05 RTIOR= 3.00

END-OF-PERIOD FLOW

HO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	HO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	.30	1	.01	.00	.01	3.	1.02	1.30	51	.08	.03	.05	68.
1.01	1.00	2	.01	.00	.01	3.	1.02	2.00	52	.08	.03	.05	87.
1.01	1.30	3	.01	.00	.01	3.	1.02	2.30	53	.08	.03	.05	99.

REC IN D:

STRTQ= -1.00 QRCSN= -.05 RTIOR= 3.00

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	END-OF-PERIOD FLOW MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	.30	1	.01	.00	.01	3.	1.02	1.30	51	.08	.03	.05	68.
1.01	1.00	2	.01	.00	.01	3.	1.02	2.00	52	.08	.03	.05	87.
1.01	1.30	3	.01	.00	.01	3.	1.02	2.30	53	.08	.03	.05	99.
1.01	2.00	4	.01	.00	.01	3.	1.02	3.00	54	.08	.03	.05	108.
1.01	2.30	5	.01	.00	.01	3.	1.02	3.30	55	.08	.03	.05	114.
1.01	3.00	6	.01	.00	.01	3.	1.02	4.00	56	.08	.03	.05	118.
1.01	3.30	7	.01	.00	.01	3.	1.02	4.30	57	.08	.03	.05	121.
1.01	4.00	8	.01	.00	.01	3.	1.02	5.00	58	.08	.03	.05	123.
1.01	4.30	9	.01	.00	.01	3.	1.02	5.30	59	.08	.03	.05	124.
1.01	5.00	10	.01	.00	.01	3.	1.02	6.00	60	.08	.03	.05	125.
1.01	5.30	11	.01	.00	.01	3.	1.02	6.30	61	.27	.22	.05	142.
1.01	6.00	12	.01	.00	.01	3.	1.02	7.00	62	.27	.22	.05	345.
1.01	6.30	13	.03	.00	.03	3.	1.02	7.30	63	.27	.22	.05	501.
1.01	7.00	14	.03	.00	.03	4.	1.02	8.00	64	.27	.22	.05	607.
1.01	7.30	15	.03	.00	.03	5.	1.02	8.30	65	.27	.22	.05	679.
1.01	8.00	16	.03	.00	.03	6.	1.02	9.00	66	.27	.22	.05	730.
1.01	8.30	17	.03	.00	.03	6.	1.02	9.30	67	.27	.22	.05	764.
1.01	9.00	18	.03	.00	.03	6.	1.02	10.00	68	.27	.22	.05	788.
1.01	9.30	19	.03	.00	.03	7.	1.02	10.30	69	.27	.22	.05	805.
1.01	10.00	20	.03	.00	.03	7.	1.02	11.00	70	.27	.22	.05	817.
1.01	10.30	21	.03	.00	.03	7.	1.02	11.30	71	.27	.22	.05	824.
1.01	11.00	22	.03	.00	.03	7.	1.02	12.00	72	.27	.22	.05	830.
1.01	11.30	23	.03	.00	.03	7.	1.02	12.30	73	1.04	.99	.05	1065.
1.01	12.00	24	.03	.00	.03	7.	1.02	13.00	74	1.04	.99	.05	1747.
1.01	12.30	25	.13	.01	.12	8.	1.02	13.30	75	1.25	1.20	.05	2457.
1.01	13.00	26	.13	.01	.12	13.	1.02	14.00	76	1.25	1.20	.05	3040.
1.01	13.30	27	.15	.01	.14	18.	1.02	14.30	77	1.56	1.51	.05	3649.
1.01	14.00	28	.15	.04	.11	33.	1.02	15.00	78	1.56	1.51	.05	4255.
1.01	14.30	29	.19	.14	.05	97.	1.02	15.30	79	1.90	1.85	.05	4837.
1.01	15.00	30	.19	.14	.05	214.	1.02	16.00	80	6.01	5.96	.05	6702.
1.01	15.30	31	.23	.18	.05	330.	1.02	16.30	81	1.46	1.41	.05	9463.
1.01	16.00	32	.73	.68	.05	585.	1.02	17.00	82	1.46	1.41	.05	9245.
1.01	16.30	33	.18	.13	.05	940.	1.02	17.30	83	1.14	1.10	.05	7932.
1.01	17.00	34	.18	.13	.05	928.	1.02	18.00	84	1.14	1.10	.05	6836.
1.01	17.30	35	.14	.09	.05	779.	1.02	18.30	85	.12	.07	.05	5733.
1.01	18.00	36	.14	.09	.05	653.	1.02	19.00	86	.12	.07	.05	4221.
1.01	18.30	37	.01	.00	.01	535.	1.02	19.30	87	.12	.07	.05	3010.
1.01	19.00	38	.01	.00	.01	385.	1.02	20.00	88	.12	.07	.05	2153.
1.01	19.30	39	.01	.00	.01	269.	1.02	20.30	89	.12	.07	.05	1574.
1.01	20.00	40	.01	.00	.01	186.	1.02	21.00	90	.12	.07	.05	1134.
1.01	20.30	41	.01	.00	.01	131.	1.02	21.30	91	.12	.07	.05	870.
1.01	21.00	42	.01	.00	.01	87.	1.02	22.00	92	.12	.07	.05	645.
1.01	21.30	43	.01	.00	.01	62.	1.02	22.30	93	.12	.07	.05	515.
1.01	22.00	44	.01	.00	.01	45.	1.02	23.00	94	.12	.07	.05	444.
1.01	22.30	45	.01	.00	.01	41.	1.02	23.30	95	.12	.07	.05	398.
1.01	23.00	46	.01	.00	.01	36.	1.03	0.00	96	.12	.07	.05	357.
1.01	23.30	47	.01	.00	.01	33.	1.03	.30	97	0.00	0.00	0.00	320.
1.02	0.00	48	.01	.00	.01	29.	1.03	1.00	98	0.00	0.00	0.00	286.
1.02	.30	49	.08	.03	.05	26.	1.03	1.30	99	0.00	0.00	0.00	257.
1.02	1.00	50	.08	.03	.05	42.	1.03	2.00	100	0.00	0.00	0.00	230.
									SUM	29.60	25.82	3.77	98987.
										(752.)	(656.)	(96.)	(2803.00)

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	9463.	5724.	1920.	989.	98968.
CMS	268.	162.	54.	28.	2800.
INCHES		17.87	23.97	25.72	25.72
MM		453.83	608.84	653.26	653.26
AC-FT		2838.	3808.	4085.	4085.
THOUS CU M		3501.	4697.	5039.	5039.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 1

3.	3.	3.	3.	3.	3.	3.	3.	3.	3.
3.	3.	3.	4.	5.	6.	6.	6.	7.	7.
7.	7.	7.	7.	8.	13.	18.	33.	97.	214.

THOUS CU M 3501. 4697. 5039.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 1

3.	3.	3.	3.	3.	3.	3.	3.	3.	3.
3.	3.	3.	4.	5.	6.	6.	6.	7.	7.
7.	7.	7.	7.	8.	13.	18.	33.	97.	214.
330.	585.	940.	928.	779.	653.	535.	385.	269.	184.
131.	87.	42.	45.	41.	36.	33.	29.	26.	42.
68.	87.	99.	108.	114.	118.	121.	123.	124.	125.
182.	345.	501.	607.	679.	730.	764.	788.	805.	817.
824.	830.	1065.	1747.	2457.	3080.	3649.	4255.	4837.	6702.
9463.	9245.	7932.	6836.	5733.	4221.	3010.	2153.	1574.	1134.
870.	645.	515.	444.	398.	357.	320.	286.	257.	230.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	9463.	5724.	1920.	989.	9886.
CMS	268.	162.	54.	28.	2800.
INCHES		17.87	23.97	25.72	25.72
MM		453.83	608.84	653.26	653.26
AC-FT		2438.	3806.	4085.	4085.
THOUS CU M		3501.	4697.	5039.	5039.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 2

1.	2.	2.	2.	2.	2.	2.	2.	2.	1.
1.	1.	2.	2.	3.	3.	3.	3.	3.	3.
3.	3.	3.	3.	4.	7.	9.	16.	48.	107.
166.	293.	470.	464.	389.	326.	267.	192.	134.	93.
65.	44.	31.	23.	20.	18.	16.	15.	13.	21.
34.	43.	50.	54.	57.	59.	60.	62.	62.	63.
91.	173.	250.	303.	340.	365.	382.	394.	402.	408.
412.	415.	532.	874.	1228.	1540.	1825.	2128.	2418.	3351.
4731.	4623.	3966.	3418.	2866.	2111.	1505.	1076.	787.	567.
435.	322.	257.	222.	199.	178.	160.	143.	128.	115.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	4731.	2862.	960.	494.	49434.
CMS	134.	81.	27.	14.	1400.
INCHES		8.93	11.99	12.86	12.86
MM		226.92	304.42	326.63	326.63
AC-FT		1419.	1904.	2043.	2043.
THOUS CU M		1750.	2348.	2520.	2520.

****

HYDROGRAPH ROUTING

ROUTE HYDROGRAPH THROUGH RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
2	1	0	0	0	0	0	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	790.	-1

STAGE	509.00	511.00	512.50	513.00	516.00
FLOW	0.00	1470.00	3350.00	4225.00	13930.00
CAPACITY=	0.	790.	1004.	1170.	1228.	1580.
ELEVATION=	485.	509.	511.	513.	513.	516.

CHEL	SPWID	COQW	EXPW	ELEVL	COOL	CAREA	EXPL
504.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

RECESSION DATA

TQ= .00 RSN= .05 STRTQ= .10

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	END-OF-PERIOD FLOW COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	.30	1	.01	.00	.01	3.	1.02	1.30	51	.08	.03	.05	68.
1.01	1.00	2	.01	.00	.01	3.	1.02	2.00	52	.08	.03	.05	87.
1.01	1.30	3	.01	.00	.01	3.	1.02	2.30	53	.08	.03	.05	99.

DAM DATA

TOPEL	COQD	EXPD	DAMWID
512.5	0.0	0.0	0.

STATION 2, PLAN 1, RATIO 1

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

1.	1.	2.	2.	2.	3.	3.	3.	3.	3.
3.	3.	3.	3.	3.	4.	4.	5.	5.	6.
6.	6.	6.	7.	7.	8.	10.	14.	26.	58.
111.	197.	339.	486.	577.	612.	607.	571.	510.	440.
370.	305.	249.	199.	161.	130.	106.	88.	73.	63.
61.	65.	72.	80.	88.	95.	101.	106.	110.	114.
124.	159.	224.	306.	390.	468.	538.	597.	646.	687.
720.	747.	797.	948.	1235.	1693.	2327.	2943.	3602.	4847.
7195.	8762.	8636.	7727.	6680.	5444.	4154.	3407.	2810.	2258.
1781.	1420.	1211.	1029.	878.	754.	650.	564.	491.	430.

STORAGE

790.	790.	790.	790.	790.	790.	790.	790.	790.	790.
790.	790.	790.	790.	791.	791.	791.	791.	791.	791.
791.	791.	791.	791.	791.	791.	791.	792.	794.	798.
806.	816.	838.	861.	874.	879.	878.	873.	864.	854.
844.	834.	824.	819.	813.	809.	805.	803.	801.	799.
799.	799.	800.	802.	803.	804.	805.	805.	806.	807.
808.	813.	823.	835.	847.	858.	868.	877.	884.	890.
895.	899.	904.	908.	970.	1024.	1080.	1134.	1187.	1251.
1336.	1393.	1388.	1359.	1317.	1272.	1223.	1174.	1122.	1074.
1032.	997.	966.	940.	918.	900.	885.	872.	862.	853.

STAGE

509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0
509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0
509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.1
509.2	509.3	509.5	509.7	509.8	509.8	509.8	509.8	509.7	509.6
509.5	509.4	509.3	509.3	509.2	509.2	509.1	509.1	509.1	509.1
509.1	509.1	509.1	509.1	509.1	509.1	509.1	509.1	509.2	509.2
509.2	509.2	509.3	509.4	509.5	509.6	509.7	509.8	509.9	509.9
510.0	510.0	510.1	510.3	510.7	511.2	511.7	512.2	512.6	513.2
513.9	514.4	514.4	514.1	513.8	513.4	513.0	512.5	512.1	511.6
511.2	510.9	510.6	510.4	510.2	510.0	509.9	509.8	509.7	509.6

PEAK OUTFLOW IS 8762. AT TIME 41.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8762.	5497.	1893.	974.	97358.
CMS	248.	156.	54.	28.	2757.
INCHES		17.16	23.64	25.33	25.33
MM		435.86	600.36	643.28	643.28
AC-FT		2726.	3755.	4023.	4023.
THOUS CU M		3362.	4631.	4962.	4962.

STATION 2, PLAN 1, RATIO 2

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

0.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	2.	2.	2.	2.	2.	3.	3.
3.	3.	3.	3.	3.	4.	5.	7.	13.	29.
[illegible]			[illegible]	[illegible]	306.	304.	285.	255.	220.

STATION 2, PLAN 1, RATIO 2

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

0.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	2.	2.	2.	2.	2.	3.	3.
3.	3.	3.	3.	3.	4.	5.	7.	13.	29.
56.	99.	169.	243.	289.	306.	304.	285.	255.	220.
185.	153.	124.	100.	80.	65.	53.	44.	36.	32.
31.	33.	36.	40.	44.	47.	50.	53.	55.	57.
62.	79.	112.	153.	195.	234.	269.	298.	323.	344.
360.	373.	399.	474.	617.	808.	1025.	1262.	1536.	2047.
2803.	3556.	3907.	3805.	3490.	3082.	2699.	2103.	1659.	1350.
1139.	950.	796.	650.	541.	453.	383.	325.	278.	239.

STORAGE

790.	790.	790.	790.	790.	790.	790.	790.	790.	790.
790.	790.	790.	790.	790.	790.	790.	790.	790.	790.
791.	790.	790.	790.	790.	791.	791.	791.	792.	794.
798.	814.	815.	825.	832.	835.	834.	832.	827.	822.
817.	812.	808.	806.	802.	799.	798.	796.	795.	795.
796.	795.	795.	796.	796.	797.	797.	798.	798.	798.
799.	802.	806.	812.	818.	824.	829.	833.	837.	840.
842.	844.	848.	859.	880.	908.	939.	974.	1010.	1055.
1122.	1184.	1207.	1200.	1179.	1146.	1104.	1060.	1021.	986.
956.	928.	904.	885.	869.	856.	846.	837.	830.	825.

STAGE

509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0
509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0
509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0	509.0
509.1	509.1	509.2	509.3	509.4	509.4	509.4	509.4	509.3	509.3
509.3	509.2	509.2	509.1	509.1	509.1	509.1	509.1	509.0	509.0
509.0	509.0	509.0	509.1	509.1	509.1	509.1	509.1	509.1	509.1
509.1	509.1	509.2	509.2	509.3	509.3	509.4	509.4	509.4	509.5
509.5	509.5	509.5	509.6	509.8	510.1	510.4	510.7	511.1	511.5
512.1	512.6	512.8	512.8	512.6	512.3	511.9	511.5	511.2	510.8
510.5	510.3	510.1	509.9	509.7	509.6	509.5	509.4	509.4	509.3

PEAK OUTFLOW IS 3907. AT TIME 41.50 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3907.	2658.	945.	496.	48593.
CMS	111.	75.	27.	14.	1376.
INCHES		8.30	11.80	12.64	12.64
MM		210.71	299.61	321.07	321.07
AC-FT		1318.	1874.	2008.	2008.
THOUS CU M		1625.	2311.	2477.	2477.

HYDROGRAPH ROUTING

CHANNEL ROUTING STN 7+00

ISTAO	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
3	1	0	0	0	0	0	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	0.	0

NORMAL DEPTH CHANNEL ROUTING

0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTOL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	0.	0

NORMAL DEPTH CHANNEL ROUTING

QN(1)	QN(2)	QN(3)	ELNVT	ELMAX	RLNTH	SEL
.0400	.0400	.0400	440.0	520.0	700.	.05000

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
3650.00 520.00 3690.00 500.00 3860.00 480.00 3960.00 460.00 4000.00 440.00
4040.00 460.00 4075.00 480.00 4100.00 500.00

STORAGE	0.00	.57	2.28	5.13	9.12	14.26	20.90	29.18	39.10	50.66
	64.03	80.28	99.60	121.99	147.43	175.24	203.69	232.71	262.31	292.47
OUTFLOW	0.00	450.35	2459.53	8430.86	18156.90	33011.36	54471.08	83808.55	122268.84	170987.38
	237603.13	324930.78	429725.27	554486.17	701414.39	881440.77	1087462.80	1315290.32	1564200.50	1833675.94
STAGE	440.00	444.21	448.42	452.63	456.84	461.05	465.26	469.47	473.68	477.89
	482.11	486.32	490.53	494.74	498.95	503.16	507.37	511.58	515.79	520.00
FLOW	0.00	450.35	2459.53	8430.86	18156.90	33011.36	54471.08	83808.55	122268.84	170987.38
	237603.13	324930.78	429725.27	554486.17	701414.39	881440.77	1087462.80	1315290.32	1564200.50	1833675.94

STATION 3, PLAN 1, RTIO 1

OUTFLOW

1.	1.	2.	2.	3.	2.	3.	3.	3.	3.
3.	3.	3.	3.	4.	4.	5.	5.	5.	6.
6.	6.	6.	6.	7.	8.	10.	13.	26.	57.
110.	194.	333.	483.	577.	611.	608.	571.	512.	441.
373.	306.	250.	200.	162.	130.	104.	88.	74.	63.
61.	65.	72.	79.	88.	94.	101.	106.	110.	114.
123.	157.	222.	304.	388.	466.	537.	595.	646.	686.
720.	746.	796.	944.	1229.	1683.	2315.	2935.	3595.	4824.
7160.	8759.	8641.	7743.	6690.	5464.	4166.	3413.	2818.	2268.
1798.	1426.	1212.	1034.	878.	757.	650.	567.	491.	432.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	1.	1.	1.	1.	1.	1.	1.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	2.	2.	3.	3.
4.	5.	5.	5.	4.	4.	3.	3.	2.	2.
2.	1.	1.	1.	1.	1.	1.	1.	1.	1.

STAGE

440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0
440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.1	440.1
440.1	440.1	440.1	440.1	440.1	440.1	440.1	440.1	440.2	440.5
441.0	441.8	443.1	444.3	444.4	444.5	444.5	444.4	444.3	444.1
443.5	442.9	442.3	441.9	441.5	441.2	441.0	440.8	440.7	440.6
440.6	440.6	440.7	440.7	440.8	440.9	440.9	441.0	441.0	441.1
441.2	441.5	442.1	442.8	443.6	444.2	444.4	444.5	444.6	444.6
444.7	444.7	444.8	445.1	445.6	446.4	447.5	448.5	449.0	449.9
451.7	452.8	452.7	452.1	451.3	450.4	449.4	448.8	448.3	447.4
446.5	445.9	445.5	445.2	445.0	444.7	444.6	444.4	444.3	444.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8759.	5496.	1493.	973.	97344.
CMS	248.	156.	54.	28.	2756.
INCHES		17.16	23.63	25.32	25.32
MM		435.80	600.29	643.19	643.19
AC-FT		3735.	2764.	4933.	4933.

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	441.1
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
451.7	452.8	452.7	452.1	451.3	450.4	449.4	448.8	448.3	447.4
444.5	445.9	445.5	445.2	445.0	444.7	444.6	444.4	444.3	444.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8759.	5496.	1893.	973.	97344.
CMS	248.	156.	54.	28.	2756.
INCHES		17.16	23.63	25.32	25.32
MM		435.80	600.29	643.19	643.19
AC-FT		2725.	3754.	4022.	4022.
THOUS CU M		3362.	4631.	4962.	4962.

MAXIMUM STORAGE = 5.

MAXIMUM STAGE IS 452.8

STATION 3, PLAN 1, RTIO 2

OUTFLOW

1.	0.	1.	1.	1.	1.	2.	1.	2.	1.
2.	1.	2.	1.	2.	2.	2.	2.	3.	3.
3.	3.	3.	3.	3.	4.	5.	7.	13.	29.
55.	97.	166.	241.	288.	306.	304.	286.	256.	221.
186.	153.	125.	100.	81.	65.	54.	44.	37.	32.
32.	32.	36.	40.	44.	47.	50.	53.	55.	57.
62.	78.	111.	152.	194.	233.	268.	298.	323.	343.
350.	373.	397.	471.	615.	804.	1022.	1257.	1531.	2035.
2790.	3549.	3904.	3809.	3493.	3089.	2606.	2112.	1664.	1354.
1141.	954.	787.	653.	542.	455.	385.	327.	280.	240.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	1.	1.	1.	1.	1.	2.
2.	3.	3.	3.	3.	2.	2.	2.	1.	1.
1.	1.	1.	1.	1.	1.	0.	0.	0.	0.

STAGE

440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0
440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.0
440.0	440.0	440.0	440.0	440.0	440.0	440.0	440.1	440.1	440.3
440.5	440.9	441.6	442.3	442.7	442.9	442.8	442.7	442.4	442.1
441.7	441.4	441.2	440.9	440.8	440.6	440.5	440.4	440.3	440.3
440.3	440.3	440.3	440.4	440.4	440.4	440.5	440.5	440.5	440.5
440.6	440.7	441.0	441.4	441.8	442.2	442.5	442.8	443.0	443.2
443.4	443.5	443.7	444.2	444.5	444.8	445.2	445.6	446.1	447.0
448.3	448.9	449.2	449.1	448.9	448.6	448.0	447.1	446.3	445.8
445.4	445.1	444.8	444.6	444.4	444.2	443.6	443.1	442.6	442.2

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3904.	2657.	945.	486.	48585.
CMS	111.	75.	27.	14.	1376.
INCHES		8.30	11.79	12.64	12.64
MM		210.70	299.57	321.02	321.02
AC-FT		1318.	1873.	2008.	2008.
THOUS CU M		1625.	2311.	2476.	2476.

MAXIMUM STORAGE = 3.

MAXIMUM STAGE IS 449.2

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STRT -1.(RECESSION DATA QRC.. =.03 RTIOR= 3.00

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	END-OF-PERIOD FLOW COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	.30	1	.01	.00	.01	3.	1.02	1.30	51	.08	.03	.05	68.
1.01	1.00	2	.01	.00	.01	3.	1.02	2.00	52	.08	.03	.05	87.
1.01	1.30	3	.01	.00	.01	3.	1.02	2.30	53	.08	.03	.05	99.

HYDROGRAPH ROUTING

CHANNEL ROUTING STN 16+00

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
4	1	0	0	0	0	0	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	0.	0

NORMAL DEPTH CHANNEL ROUTING

QN(1)	QN(2)	QN(3)	ELMVT	ELMAX	RLNTH	SEL
.0400	.0400	.0400	385.0	440.0	900.	.06100

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC

5700.00 440.00 5820.00 420.00 5880.00 400.00 6000.00 385.00 6390.00 396.00
6430.00 397.00 6490.00 400.00 6590.00 420.00

STORAGE	0.00	3.76	15.05	33.85	60.20	93.34	130.21	168.52	208.21	249.29
	291.76	335.61	380.84	427.32	474.83	523.39	572.98	623.61	675.29	728.00
OUTFLOW	0.00	2140.72	13592.70	40075.81	86046.74	166690.81	279970.06	419867.17	584224.90	772413.09
	984043.77	1218969.78	1477031.67	1763469.21	2074296.62	2408704.46	2766503.24	3147585.42	3551896.55	3979420.04
STAGE	385.00	387.49	390.79	393.68	396.58	399.67	402.37	405.26	408.16	411.05
	413.95	416.84	419.74	422.63	425.53	428.42	431.32	434.21	437.11	440.00
FLOW	0.00	2140.72	13592.70	40075.81	86046.74	166690.81	279970.06	419867.17	584224.90	772413.09
	984043.77	1218969.78	1477031.67	1763469.21	2074296.62	2408704.46	2766503.24	3147585.42	3551896.55	3979420.04

STATION 4, PLAN 1, RTIO 1

OUTFLOW

2.	0.	3.	1.	3.	2.	3.	3.	3.	3.
3.	3.	3.	3.	3.	4.	4.	5.	5.	6.
6.	6.	6.	7.	7.	8.	9.	13.	25.	55.
107.	190.	326.	477.	575.	610.	609.	573.	514.	446.
376.	309.	252.	203.	163.	132.	108.	89.	73.	64.
61.	65.	71.	79.	87.	94.	100.	106.	110.	114.
123.	155.	219.	300.	384.	463.	534.	593.	644.	685.
719.	745.	792.	935.	1214.	1660.	2290.	2929.	3569.	4791.
7084.	8756.	8650.	7776.	6708.	5504.	4189.	3427.	2833.	2280.
1911.	1435.	1221.	1040.	885.	761.	656.	569.	495.	433.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	1.	1.	1.	1.	1.	0.
1.	1.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
1.	1.	1.	1.	1.	1.	1.	1.	1.	0.
9.	10.	10.	2.	2.	3.	4.	5.	5.	6.
3.	3.	2.	2.	2.	1.	1.	1.	4.	4.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	2.	2.	3.	4.	5.	5.	6.
9.	10.	10.	9.	8.	7.	6.	5.	4.	4.
3.	3.	2.	2.	2.	1.	1.	1.	1.	1.

STAGE

385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0
385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0
385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.1
385.1	385.3	385.4	385.6	385.8	385.8	385.8	385.8	385.7	385.6
385.5	385.4	385.3	385.3	385.2	385.2	385.1	385.1	385.1	385.1
385.1	385.1	385.1	385.1	385.1	385.1	385.1	385.1	385.1	385.2
385.2	385.2	385.3	385.4	385.5	385.6	385.7	385.8	385.9	385.9
386.0	386.0	386.1	386.3	386.6	387.2	387.9	388.1	388.3	388.6
389.1	389.6	389.5	389.3	389.0	388.7	388.4	388.2	388.1	387.9
387.4	386.9	386.7	386.4	386.2	386.0	385.9	385.8	385.7	385.6

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8756.	5495.	1892.	973.	97326.
CMS	248.	156.	54.	28.	2756.
INCHES		17.15	23.63	25.32	25.32
MM		435.72	600.18	643.07	643.07
AC-FT		2725.	3754.	4022.	4022.
THOUS CU M		3361.	4630.	4961.	4961.

MAXIMUM STORAGE = 10.

MAXIMUM STAGE IS 389.6

STATION 4, PLAN 1, RTIO 2

OUTFLOW

1.	0.	1.	1.	1.	1.	2.	1.	2.	1.
2.	1.	1.	2.	2.	2.	2.	2.	3.	3.
3.	3.	3.	3.	3.	4.	5.	7.	12.	28.
53.	95.	163.	238.	287.	305.	304.	287.	258.	223.
187.	155.	126.	101.	82.	66.	54.	44.	37.	32.
30.	32.	36.	40.	44.	47.	50.	53.	55.	57.
61.	77.	109.	150.	192.	231.	266.	296.	322.	342.
359.	373.	396.	467.	608.	795.	1012.	1247.	1518.	2006.
2777.	3527.	3908.	3810.	3507.	3095.	2623.	2120.	1693.	1355.
1154.	955.	800.	653.	550.	455.	391.	326.	284.	240.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	1.	1.	1.
1.	1.	1.	1.	1.	1.	2.	2.	3.	4.
4.	5.	6.	5.	5.	5.	4.	4.	3.	2.
2.	2.	1.	1.	1.	1.	1.	1.	0.	0.

STAGE

385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0
385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0
385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0	385.0
385.1	385.1	385.2	385.3	385.4	385.4	385.4	385.4	385.3	385.3
385.3	385.2	385.2	385.1	385.1	385.1	385.1	385.1	385.0	385.0
385.0	385.0	385.0	385.1	385.1	385.1	385.1	385.1	385.1	385.1
385.1	385.1	385.1	385.2	385.3	385.3	385.4	385.4	385.4	385.5
385.5	385.5	385.5	385.6	385.8	386.1	386.4	386.7	387.1	387.7
388.1	388.2	388.3	388.3	388.2	388.1	388.0	387.9	387.3	386.8

385.3	385.2	385.2	385.1	385.1	385.1	385.1	385.1	385.0	385.0
385.0	385.0	385.0	385.1	385.1	385.1	385.1	385.1	385.1	385.1
385.1	385.1	385.1	385.2	385.3	385.3	385.4	385.4	385.4	385.5
385.5	385.5	385.5	385.6	385.8	386.1	386.4	386.7	387.1	387.7
386.1	386.2	386.3	386.3	386.2	386.1	386.0	387.9	387.3	386.8
386.6	386.3	386.1	385.9	385.7	385.6	385.5	385.4	385.4	385.3

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3908.	2657.	944.	486.	48575.
CMS	111.	75.	27.	14.	1375.
INCHES		8.29	11.79	12.64	12.64
MM		210.67	299.51	320.95	320.95
AC-FT		1318.	1873.	2007.	2007.
THOUS CU M		1625.	2310.	2476.	2476.

MAXIMUM STORAGE = 6.

MAXIMUM STAGE IS 388.3

HYDROGRAPH ROUTING

CHANNEL ROUTING STN 26+00

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
5	1	0	0	0	0	0	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPHP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	0.	0

NORMAL DEPTH CHANNEL ROUTING

QN(1)	QN(2)	QN(3)	ELNVT	ELMAX	RLNTH	SEL
.0400	.0400	.0400	359.0	420.0	1000.	.02600

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC

4700.00 420.00 4790.00 400.00 5700.00 380.00 6000.00 359.00 6180.00 365.00
6370.00 360.00 6410.00 361.00 6510.00 380.00

STORAGE	0.00	9.03	39.34	80.35	125.98	176.24	231.12	291.28	361.30	442.10
	533.45	635.98	749.07	872.67	1000.22	1129.83	1258.51	1389.25	1521.05	1653.93
OUTFLOW	0.00	3017.07	22688.84	68963.55	136434.08	224631.62	333779.12	474041.78	652140.54	862110.94
	1107520.77	1391495.73	1716919.48	2092082.88	2530471.94	3011639.75	3533133.31	4093701.48	4692167.50	5327553.71
STAGE	359.00	362.21	365.42	368.63	371.84	375.05	378.26	381.47	384.68	387.89
	391.11	394.32	397.53	400.74	403.95	407.16	410.37	413.58	416.79	420.00
FLOW	0.00	3017.07	22688.84	68963.55	136434.08	224631.62	333779.12	474041.78	652140.54	862110.94
	1107520.77	1391495.73	1716919.48	2092082.88	2530471.94	3011639.75	3533133.31	4093701.48	4692167.50	5327553.71

STATION 5, PLAN 1, RTIO 1

OUTFLOW

4.	0.	2.	2.	2.	3.	3.	3.	3.	3.
3.	3.	3.	3.	3.	4.	4.	5.	5.	6.
6.	6.	6.	7.	7.	8.	9.	13.	24.	52.
103.	183.	314.	467.	570.	609.	610.	577.	519.	450.

1107520.77 1391495.73 1716919.48 2092082.88 2530[illegible] 30 .75 3133 4093 .8 67.55 [illegible]

STATION 5, PLAN 1, RTIO 1

OUTFLOW

4.	0.	2.	2.	2.	3.	3.	3.	3.	3.
3.	3.	3.	3.	3.	4.	4.	5.	5.	6.
6.	6.	6.	7.	7.	8.	9.	13.	24.	52.
103.	183.	314.	467.	570.	609.	610.	577.	519.	450.
380.	316.	265.	204.	165.	134.	109.	90.	74.	64.
61.	46.	71.	79.	86.	94.	100.	105.	110.	114.
122.	152.	213.	294.	378.	458.	529.	589.	640.	682.
714.	744.	788.	921.	1190.	1622.	2239.	2887.	3553.	4720.
6996.	8724.	8684.	7807.	6755.	5546.	4244.	3433.	2878.	2316.
1843.	1458.	1231.	1056.	893.	771.	661.	575.	500.	438.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	1.	1.	1.	2.	2.	2.	2.	2.	1.
1.	1.	1.	1.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	1.	1.	2.	2.	2.	2.
2.	2.	2.	3.	4.	5.	7.	9.	10.	12.
15.	18.	18.	16.	15.	13.	11.	10.	9.	7.
6.	4.	4.	3.	3.	2.	2.	2.	1.	1.

STAGE

359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0
359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0
359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.1
359.1	359.2	359.3	359.5	359.6	359.6	359.6	359.6	359.6	359.5
359.4	359.3	359.3	359.2	359.2	359.1	359.1	359.1	359.1	359.1
359.1	359.1	359.1	359.1	359.1	359.1	359.1	359.1	359.1	359.1
359.1	359.2	359.2	359.3	359.4	359.5	359.6	359.6	359.7	359.7
359.8	359.8	359.8	360.0	360.3	360.7	361.4	362.1	362.3	362.5
362.9	363.1	363.1	363.0	362.8	362.6	362.4	362.3	362.1	361.5
361.0	360.6	360.3	360.1	360.0	359.8	359.7	359.6	359.5	359.5

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8724.	5494.	1892.	973.	97295.
CMS	247.	156.	54.	28.	2755.
INCHES		17.15	23.62	25.31	25.31
MM		435.64	600.00	642.87	642.87
AC-FT		2726.	3752.	4020.	4020.
THOUS CU M		3361.	4629.	4959.	4959.

MAXIMUM STORAGE = 18.

MAXIMUM STAGE IS 363.1

STATION 5, PLAN 1, RTIO 2

OUTFLOW

2.	0.	1.	1.	1.	1.	1.	2.	1.	2.
1.	2.	1.	2.	2.	2.	2.	2.	3.	3.
3.	3.	3.	3.	3.	4.	5.	6.	12.	26.
51.	91.	157.	234.	284.	305.	305.	289.	260.	225.
190.	157.	129.	103.	83.	67.	55.	45.	37.	32.
30.	32.	35.	39.	43.	47.	50.	53.	55.	57.
61.	76.	107.	147.	189.	229.	264.	294.	320.	341.
358.	372.	394.	459.	595.	781.	996.	1230.	1497.	1960.
2714.	3510.	3897.	3827.	3513.	3117.	2659.	2157.	1720.	1377.
1166.	974.	805.	668.	552.	466.	391.	334.	283.	246.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	1.	1.	1.	1.	1.	1.	1.
1.	0.	0.	0.	0.	0.	0.	0.		
0.	0.	0.							

81.	76.	107.	147.	199.	274.	2[illegible].	[illegible].		
8.	72.	394	45			5.	30.	1497	196
2716.	3510.	3897.	3827.	3513.	3117.	2659.	2157.	1720.	1377.
1166.	974.	805.	668.	552.	466.	391.	334.	283.	246.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	1.	1.	1.	1.	1.	1.	1.
1.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	2.	2.	3.	4.	4.	6.
8.	10.	10.	10.	10.	9.	8.	6.	5.	4.
3.	3.	2.	2.	2.	1.	1.	1.	1.	1.

STAGE

359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0
359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0
359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0	359.0
359.1	359.1	359.2	359.2	359.3	359.3	359.3	359.3	359.3	359.2
359.2	359.2	359.1	359.1	359.1	359.1	359.1	359.0	359.0	359.0
359.0	359.0	359.0	359.0	359.0	359.0	359.1	359.1	359.1	359.1
359.1	359.1	359.1	359.2	359.2	359.2	359.3	359.3	359.3	359.4
359.4	359.4	359.4	359.5	359.6	359.8	360.1	360.3	360.6	361.1
361.9	362.3	362.4	362.3	362.3	362.2	361.8	361.3	360.8	360.5
360.2	360.0	359.9	359.7	359.6	359.5	359.4	359.4	359.3	359.3

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3897.	2656.	944.	496.	48558.
CMS	110.	75.	27.	14.	1375.
INCHES		8.29	11.79	12.63	12.63
MM		210.60	299.41	320.84	320.84
AC-FT		1317.	1872.	2007.	2007.
THOUS CU M		1625.	2310.	2475.	2475.

MAXIMUM STORAGE # 10.

MAXIMUM STAGE IS 362.4

********** ********** ********** ********** **********

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS	
				RATIO 1 1.00	RATIO 2 .50
HYDROGRAPH AT	1	2.98 (7.72)	1	9461. (267.96)	4731. (133.98)
ROUTED TO	2	2.98 (7.72)	1	8762. (248.11)	3907. (110.62)
ROUTED TO	3	2.98 (7.72)	1	8759. (248.03)	3904. (110.55)
ROUTED TO	4	2.98 (7.72)	1	8756. (247.93)	3904. (110.67)
ROUTED TO	5	2.98 (7.72)	1	8724. (247.04)	3897. (110.34)

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	509.00	509.00	512.50
STORAGE	790.	790.	1170.
OUTFLOW	0.	0.	3350.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	514.40	1.90	1393.	8762.	5.00	41.00	0.00
.50	512.82	.32	1207.	3907.	2.00	41.50	0.00

PLAN 1 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	8759.	452.8	41.00
.50	3904.	449.2	41.50

PLAN 1 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	8756.	389.6	41.00
.50	3908.	388.3	41.50

PLAN 1 STATION 5

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	8724.	363.1	41.00
.50	3897.	362.4	41.50

STABILITY ANALYSIS

APPENDIX E

TAMS

Job No. 1551-08 Sheet 1 of 11
Project NYS Dam Inspections - Tivoli Date May 29, 12
Subject Structural Stability - Phase 1 By JJT
Ch'k. by

Assumptions

1) The Unit Weight of concrete and backfill within the dam are assumed to be 150 lbs/cu ft. & 120 lbs/cu ft.
2) Ice load of 5000 lbs/sq ft acting about 1 ft. from top of dam. (COE Criteria)
3) All forces due to the deck and loading of the deck are considered to be equally transferred to the base.
4) Angle of internal resistance of Foundation material is assumed to be 35°
5) Dam site is in Seismic zone 2

Loading Conditions

Case I - Normal Loading: Lake level at Spillway Crest El. 599 No Ice Load

Case II - Normal Loading: Lake level at Spillway Crest El. 599, With ice load

Case III - Unusual loading: Lake level at ½PMF

Case IV - Extreme loading: Lake level at PMF

Case V - Unusual loading: Lake level at Spillway Crest and earthquake force of 0.05

Stability Criteria

a) Overturning: Resultant Forces shall fall within the middle third of the base for Case I and II and the resultant force shall fall within the middle half of the base for cases III, IV and V.

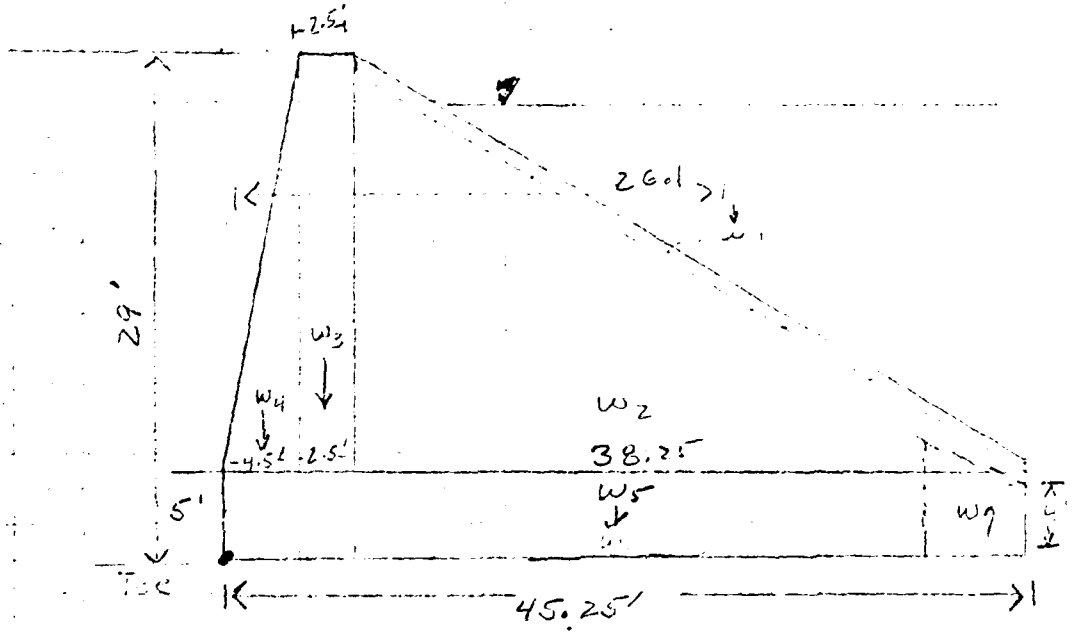
b) Sliding. For Case I and II, friction factor of safety is to be 1.5
For cases III & V, F.F.S = 1.25, For Case IV FFS = 1.1

TAMS

Job No. 155[illegible]
Project NYS Dam Inspections - Tivoli
Subject Structural Stability - Phase I

Sheet 2 of [illegible]
Date [illegible]
By JF
Ch'k. by

Dead Loads



Scale 1" = 10'

W_1 = weight of 1 Bay (15 ft of Decking)

15 ft × 0.75' × 44' (150 pcf) = 74.25/1.33 = 55.7 Kips

$W_2 = \frac{1}{2}(38.25)(24')(0.150)\frac{(133)}{1.33}$ = 68.85 "

$W_3 = 2.5(24)(0.150)\frac{(1.33)}{1.33}$ = 9.0 "

$W_4 = \frac{1}{2}(4.5)\,24\,(0.150)\frac{(1.33)}{1.33}$ = 8.1 "

$W_5 = 5(45.25)(0.150)\frac{(1.33)}{1.33}$ = 33.9

W_7 = weight of Concrete "Key"

$\left(\frac{4'6}{2}\right)\frac{(2.5')(0.150)(13.67')}{1.33}$ = $\frac{55.9}{23.44}$

TAMS

Job No. 1551-03 Sheet 3 of ____

Project Nys Dam Insp. - Tivoli Date ____

Subject Structural Stability - Phase I By JJF

Ch'k. by ____

ΣM about Toe

$\Sigma M_y = w_1(26.1) + w_2(19.75) + w_3(5.75) + w_4(3.0) + w_5(22.6) + w_6(41.6)$

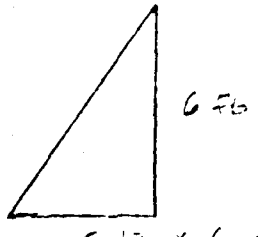
$\Sigma M_y = 5976.4$

$\bar{x} = \frac{5976.4}{231.44} = 25.8$

$\Sigma M_y = w_1(17) + w_2(13) + w_3(17) + w_4(13.0) + w_5(2.5) + w_6(2.5$

$\bar{y} = \frac{2324.7}{231.44} = 10.04$

Passive Resistance of Backfill Between Bays



$0.12 \times 6 = 0.72$ Kip

$0.72 \times \frac{6}{2} \quad \frac{13.67 \text{ ft}}{1.33 \text{ ft}} = 22.20 \leftarrow$ (k) $\times 2$ (DA) $= 44.4$ (k/c)

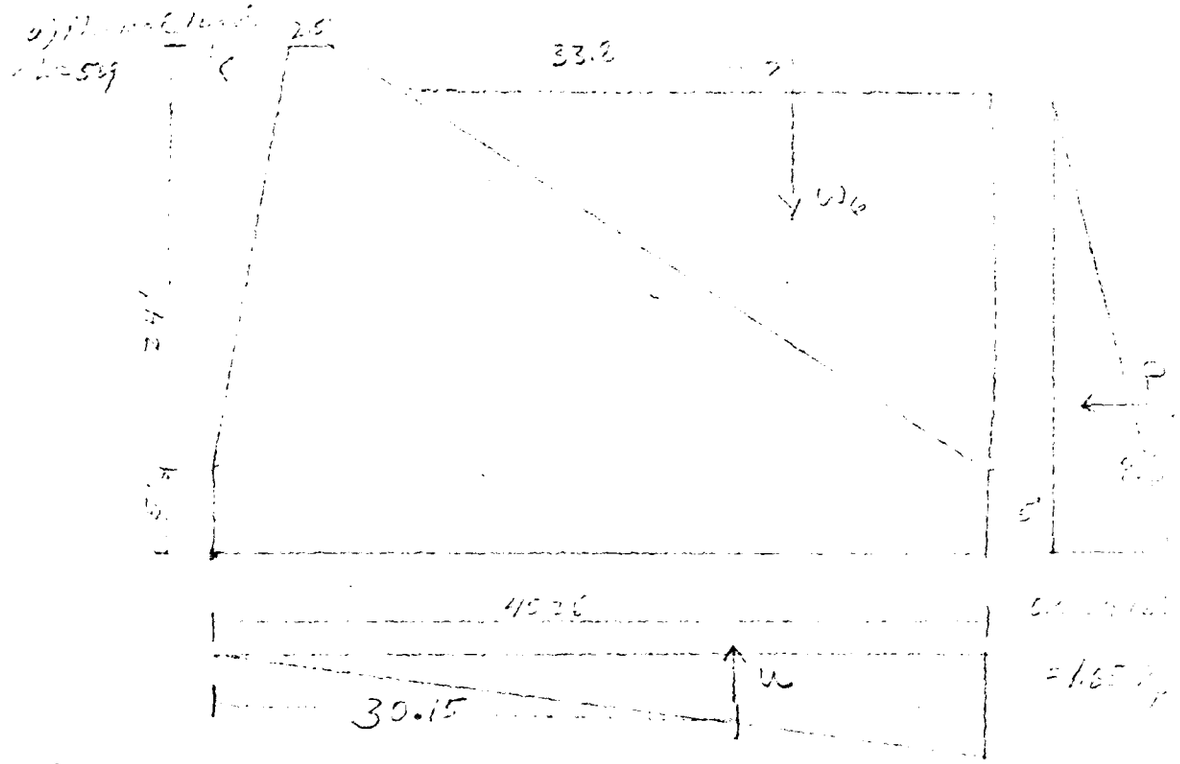
TAMS

Job No. _____ Sheet _____ of _____

Project _____ Date _____

Subject _____ By _____

Ch'k. by _____



Σ M @ TOE

$P = \frac{1}{2}\left(\frac{1.65}{1.33}\right)(24)(15) = 245.6 \times 13.8$

$U = \frac{1}{2}(1.65)(4.525)\left(\frac{1.33}{1.33}\right) = 37.3 \times 30.15$

$W = \frac{1}{2}(22)(24.5)(0.0624) = 273.24 \times 33.8$

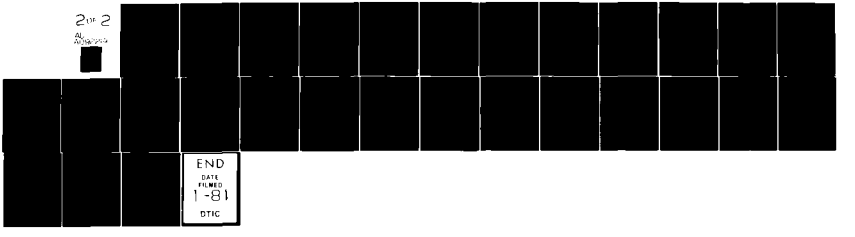
$F_V = \downarrow 235.14\ K$

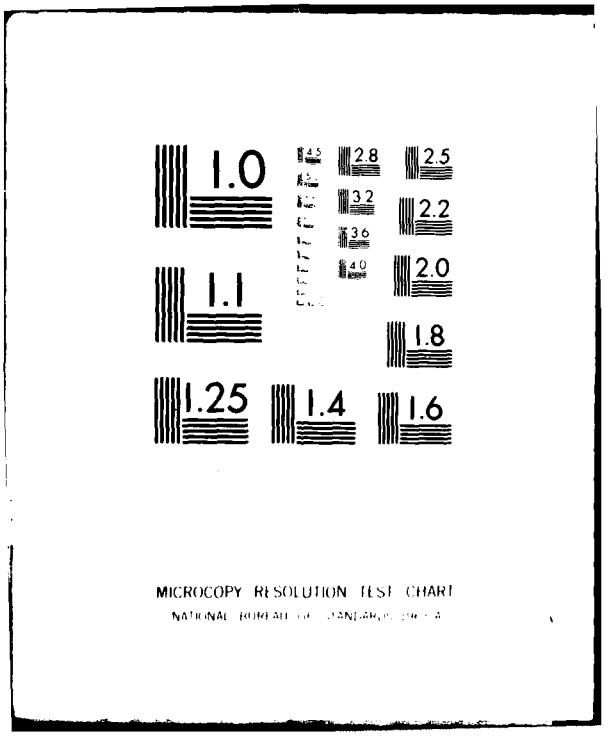
$F_H = \leftarrow 245.6\ K$

$M_R = 9235.5\ kF$

$M_O = 3297.1\ kF$

AD-A092 219 NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/13
NATIONAL DAM SAFETY PROGRAM. TIVOLI LAKE DAM (INVENTORY NUMBER --ETC(U)
SEP 80 E O'BRIEN DACW51-79-C-0001
UNCLASSIFIED NL




1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

TAMS

Job No. 1531-08 Sheet 5 of 11

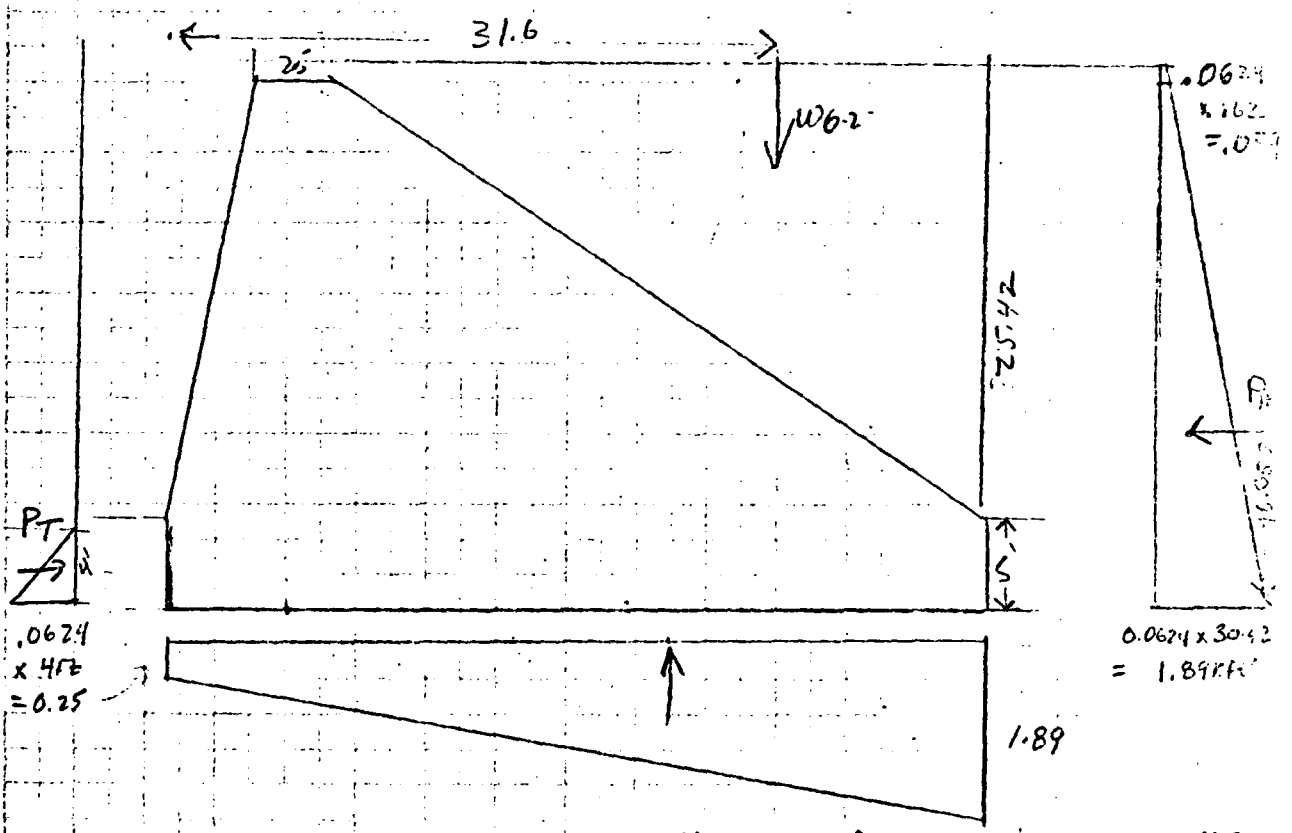
Project NYS Dam Insp. - Tivoli Date May 29, 19[illegible]

Subject Structural Stability By JMF

Ch'k. by

Unusual Loading - ½ PMF

WL @ EL 513.12 ; Tailwater 491.12



	K	MA	Mo	MR
$P_H = \left[\frac{1.89 + .039}{2}\right](29.8)\frac{(15)}{1.33}$	324.06	10.06	3260.04	
$P_T = \frac{1}{2}(0.25)(4)(0.6)\frac{(1.33)}{1.33}$	0.3	1.33		0.4
$U = \frac{1.89 + 0.25}{2}(45.25)\frac{1.33}{1.33}$	48.42	28.34	1372.2	
$W_6 = \frac{1}{2}(25.42)(38.25)(0.0624)\frac{(15)}{1.33}$	350	32.5		11376.

$F_V\downarrow$ = 301.58

$F_H\leftarrow$ = 323.66

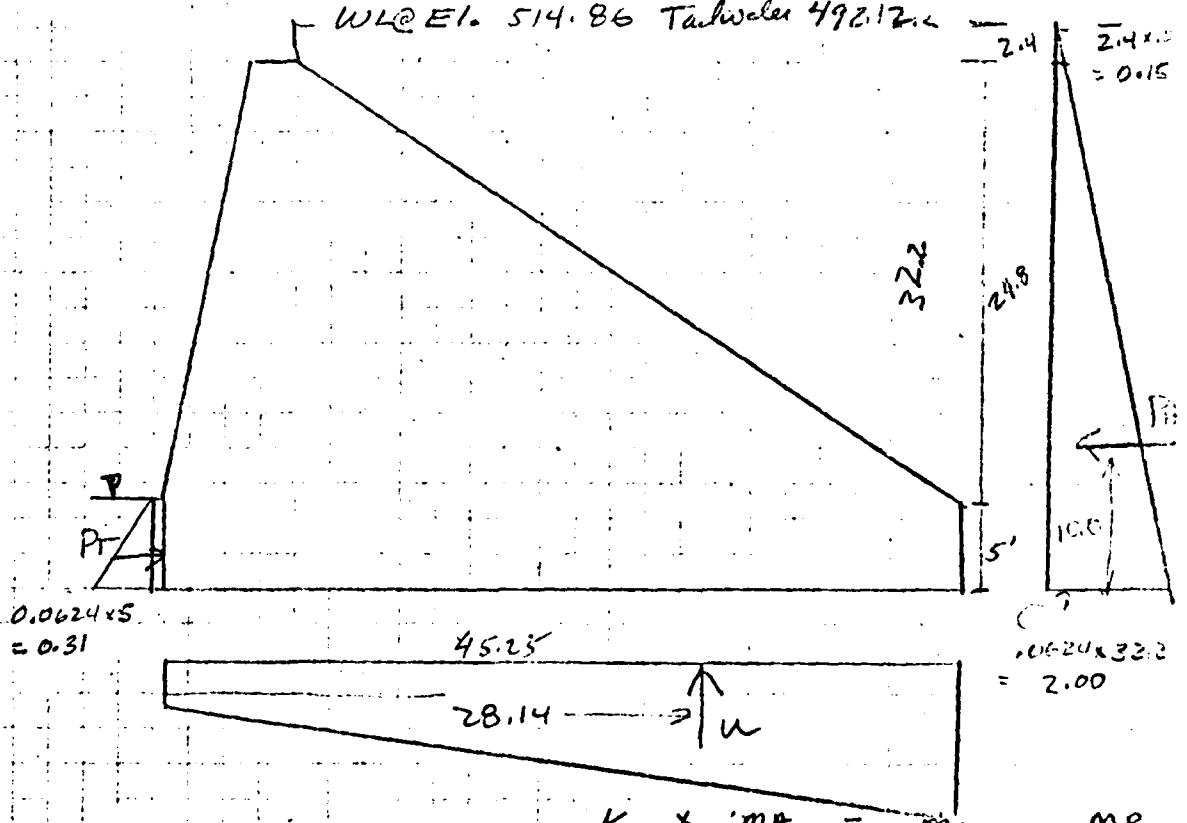
M_R = 11376.4

M_O = 4632.24

TAMS

Job No. 1551-08 Sheet 6 of 11
Project NYS DAM INSP. TIVOLI Date May 29 19
Subject STRUCTURAL STABILITY By TJF
Ch'k. by

Extreme LOADING - PMF



		K	×	MA	=	Mo	MR
$P_H = \frac{2.00+0.15}{2}(32.2)\frac{15}{1.33}$	=	390.4		10.6		4138.2	
$P_T = \frac{1}{2}(5)(.31)(0.6)\frac{1.33}{1.33}$	= 0.47		× 1.67				0.78
$U = \frac{2.00+0.31}{2}(45.25)\frac{1.33}{1.33}$	= 52.26		×			1470.7	
$W_o = \frac{\frac{1}{2}(27.2)(15)(38.25)(0.0624)}{1.3}$	= 374.5		×	32.5			12171.25

$F_V\downarrow$ 322.24
$F_H\leftarrow$ 389.9
MR = 12172.03
Mo = 5608.9

TAMS

Job No. 1551-08 Sheet 7 of 11
Project NYS Dam Insp. Tivoli Date 5-30-80
Subject Structural Stability By JJF
Ch'k. by

Ice Load - Per bay

$$\frac{15.0 \text{ ft } (5.0)}{1.33} = \underset{\leftarrow}{\overset{K}{56.39}} \times \overset{MA}{26.3} = \overset{Mo}{1483.1}$$

Case I - Normal loading - No ice

	F_V	F_H	M_R	M_o
Dead Load	231.44	-22.2	6020.8	
Hydrostatic	235.9 / 467.3	245.6 / 223.4	9235.5 / 15236.3	3287.1 / 3287.1

$$EM = 15236.3 - 3287.1 = 11969.2$$

$$\bar{x} = \frac{11969.2}{467.3} = 25.61 \qquad \text{OK - inside center } 1/3$$

$$\bar{e} = 22.75 - 25.61 = -2.86 \text{ (upstream of } \ell)$$

$$\bar{p} = \frac{467.3}{45.25}\left(1 \pm \frac{6 \times 2.86}{45.25}\right)\frac{1000}{144} = 71.7 \pm 27.2 = 98.9 \text{ psi heel}$$
$$= 44.5 \text{ psi toe}$$

Friction Factor of Safety

$$\frac{470 \tan 35^\circ}{220} = 1.5 = 1.5 \quad \text{OK}$$

TAMS

Job No. 1551-08 Sheet 8 of 11
Project NYS Dam Insp - Tivoli Date May 20 1986
Subject Structural Stability By JJF
Ch'k. by

II - Normal Loading with Ice load

	F_V	F_H	M_R	M_O
Dead Load	231.44	-22.2	6020.8	
Hydrostatic	235.9	245.6	9235.5	3287.1
Ice Load		56.4		1483.1
	467.3	279.8	15236.3	4770.2

$\Sigma M = 15236.3 - 4770.2 = 10466.1$

$\bar{x} = \frac{10466.1}{467.3} = 22.4$

$\bar{e} = 22.75 - 22.4 = 0.35$ Ok inside center 1/3

$\bar{p} = \frac{467.3}{45.25}\left(1 \pm \frac{6 \times 0.35}{45.25}\right)\frac{1000}{144} = 71.7 \pm 3.3$ 75 psi toe, 68.4 hel

Friction Factor of Safety

$\frac{470 \tan 35^\circ}{279.8} = 1.2 < 1.5$ No good

TAMS

Job No. 1551-08 Sheet 9 of 11
Project NYS Dam Inspection - Tivoli Date May 30 1980
Subject Structural Stability By JJF
Ch'k. by

Case III - Unusual Loading ½ PMF

	F_V	F_H	M_R	M_O
Dead Load	231.44	-22.2	6020.8	
Hydrostatic	301.58	323.66	11376.4	4632.24
	533.0	301.46	17397.2	4632.24

$$\Sigma M = 17397.2 - 4632.24 = 12765$$

$$\bar{x} = \frac{12765}{533} = 23.95$$

OK - inside center ½

$$\bar{e} = 22.75 - 23.95 = -1.20 \text{ upstream}$$

$$\bar{P} = \frac{533.02}{45.25}\left(1 \pm \frac{6 \times 1.20}{45.25}\right)\frac{1000}{144} = 81.80 \pm 13.0 = 94.8 \text{ psi heel}, \ 68.8 \text{ " Toe}$$

Friction Factor of Safety

$$\frac{533.0 \ \tan 35^\circ}{301.46} = 1.25 \quad \text{OK}$$

TAMS

Job No. 1551-08 Sheet 10 of 11
Project NYS Dam Insp. - Tivoli Date May 30, 1990
Subject Structural Stability By JJF
Ch'k. by

Case IV - Extreme loading - PMF

	F_V	F_H	M_R	M_O
Dead Load	231.44	-22.2	6020.8	
Hydrostatic Load	322.24	389.9	12172.0	5609.9
	553.68	367.7	18,192.8	5,608.9

$\Sigma M = 18,192.8 - 5608.9 = 12583.9$ OK-inside center 1/2

$\bar{N} = \frac{12583.9}{553.68} = 22.72$

$\bar{e} = 22.25 - 22.72 = 0.50$

$\bar{p} = \frac{553.68}{45.25}\left(1 \pm \frac{6 \times 0.50}{46.25}\right)\frac{1000}{144} = 85.0 \pm 5.6 = 90.6 \text{ psi} - \text{A}$
$= 79.4 \text{ psi} - \text{T}$

Friction Factor of Safety

$\frac{555}{370} \quad \frac{\text{Tan } 35^\circ}{} = 1.05 < 1.1$

TAMS

Job No. 1551-08 Sheet 11 of 11
Project NYS Dam Insp - Tivoli Date 5-30-80
Subject Structural Stability By JJF
Ch'k. by

Case V - Unusual Loading Earthquake Force = 0.05g

Zone 2 0.05g
Zangers Coef C = 0.27

① Hydrodynamic Force

$P = 0.27 \times 0.05 \times 0.0624 \times 26.3^2 \frac{(15)}{1.33} = 6.57$

$M_p = 6.57\,(26.3)(0.4) = 69.1$ kF

② Dynamic Force

$W_D = 0.05\,(231.44) = 11.57$

$M_{WD} = 11.57\,(10.04) = 116.16$ kF

	F_V	F_H	M_R	M_O
Dead Load	231.44	-22.2	6020.8	
Hydrostatic	235.9	245.6	9235.5	3287.1
Hydrodynamic		6.57		69.1
Dynamic		11.57		116.16
	467.3	241.54	15236.3	3472.36

$\varepsilon_m = 15236.3 - 3472.36 = 11763.94$

$\bar{x} = \frac{11763.94}{467.3} = 25.17$ OK - within middle 1/2

$\bar{e} = 22.25 - 25.17 = -2.92$

$\bar{p} = \frac{467.3}{45.25}\left(1 \pm \frac{6 \times 2.92}{45.25}\right)\frac{1000}{144} = 71.7 \pm 27.8 =$ 99.5 psi Heel, 43.9 psi Toe

Friction Factor of Safety

$\frac{467.3 \tan 35}{241.54} = 1.36 > 1.3$ OK

OTHER DATA

APPENDIX F

Report on Rehabilitation to Cranberry Dam
Pothat Water Company
Sloatsburg, New York

November 8, 1974

A hollow concrete dam is situated at the easterly end of 100 acre Cranberry Lake. Dam is approximately 135' long x 25' high (estimated) and was constructed with Reinforced Concrete. Dam is constructed with the intent to impound water only since there are no provisions to permit the flow of water over the dam. The Lake had been provided with a spillway structure of ample size to handle the highest recorded runoffs without any threat of water level rise above the Dam Crest.

Upstream end of the dam is a 9" thick sloped concrete deck supported by horizontal beams which were tied into the reinforced buttresses. There are 9 buttresses, 8 bays located at approximately 15-ft. on centers. Downstream end of the dam is open and buttress foundations follow the original sloping lines of the ground. The dam stretches between two concrete retaining walls.

At the time of inspection, the water level was being lowered through 8" blow off line. There was no evidence of running leaks thru the concrete deck except for slow seepages at the junction points of the buttresses to the horizontal beams, and on few isolated places along the surfaces of the buttresses. The ground in bays 6, 7, and 8 were saturated as a direct result of leakage thru a separated joint between the dam structure and the wing wall at the south end. Free flowing water at the same type of joint in the northerly end had been observed prior to dropping of lake level. However, present draw down is sufficient enough not to produce any leak. There are evidences of past repairs along these joints but existing conditions dictate the undertaking of diligent corrective measures.

There are a variety of spalls in the concrete mainly concentrated along the bottoms and faces of horizontal beams and very little amount along the bottom of sloped deck and the faces of the buttresses. These spalds are not serious in nature but needs certain repair work. On the south side of northerly Bay 1, there is a longitudinal crack in the deck running along the inside face of the buttress. Concrete at this location spallded considerably and the contractor removed the loose concrete to a depth of 4" in an area confined to approximately 3 to 4 square feet. Instructions as outlined below were given to the contractor to proceed with the repair.

The upstream face of the sloped concrete deck has had certain repair work done in the past, but to date some of these failed thus, contributing to the slow progressive deterioration of the deck surface and if not corrected, will continue to cause added problems in the future.

A majority of the seepage thru hairline cracks underneath the dam was plugged by the calcium deposits formed over a period of years and continuance of the process is still in evidence.

Basically, structural integrity of the concrete is sound and does not render itself to concern. Specifications prepared by Mr. Remig A. Papp, Consulting Engineer, coupled with the recommendations mentioned in this report will ensure the proper rehabilitation of the subject dam.

Recommendations:

Upstream end of the Dam

1. Remove the hollow concrete patches over bays 5 and 6 and replace them with low slump concrete preferably made with Antihydro Cement and 3/8" pea gravel. Apply 1 coat of Uniweld or Sika Dur Hi-Mod over old concrete to insure the proper bonding.

2. Repeat process one for section over Bay 1.

3. Chip out cracked concrete along Bays 7 and 8, apply one coat Colma Joint Primer and fill with Colma Joint Sealer (As manufactured by Sika)

4. All joints between concrete slabs exhibit cracks thru tar joints. These joints should be sealed with Colma Joint Sealer after application of primer.

5. Excavate earth to a depth of 4' to 5' below water level along the two wing walls and clean out the joint between wall and the dam to sound surfaces. Follow process of sealing as mentioned in items 3 and 4. Backfill with impervious select fill and compact it to a level of high water mark. Recommended use of Igas Joint Sealer should be eliminated.

6. Follow manufacturer's recommendations for preparing surfaces and application of joint material. Drying of concrete surfaces can be aided by low intensity flame application.

7. Apply Bentonite at a rate of 75 lbs. per 100 S.F. for entire length of the dam with a width of approximately 25' out into the lake from the existing water line on the deck.

8. There is no need of lowering water level in the lake any further.

Underneath - Downstream of the Dam

1. Sound all concrete surfaces where spalling, scaling, rusting or other evidence of deterioration are present. Remove all loose concrete and patch according to recommendations outlined by Mr. Papp.

2. Form and place concrete underneath the sloped and cracked deck of Bay 1 to provide added strength and support to the deck. Cracked concrete should be repaired with Sika Dur. Hi-Mod and application of low slump nonshrink grout made with antihydro cement. Concrete support should be mounded with the available fill of the site to eliminate the effect of frost action.

3. Do not remove calcium deposits over hairline cracks or elsewhere for patch work repairs unless the concrete exhibits deterioration.

4. Remove rotten wood forms left in the concrete and patch them as recommended.

Gungor Bastug

Prepared by: Gungor Bastug, C.E
Hackensack Water Co.

GB/aj

476 Lookout Avenue
Hackensack, New Jersey 07601
January 25, 1975

Pothat Water Company
Post Office Box 45
Sloatsburg, New York 10974

Attention: Mr. S. A. Wyle,
Vice President and
Secretary-Treasurer

Re: Rehabilitation Of
Cranberry Dam And
Spillway

Gentlemen:

I am submitting this final report to you, together with a "marked up" drawing to delineate the work performed on the above subject matter and also the comments on the results of such work based upon my field observations during and after the completion of the proposed work.

Recommendations as mentioned in my November 8, 1974, report and suggested method of plugging leaks by Mr. Remig A. Popp formed the general outline of this undertaking, which was carried out by Contractor Miles Kuchar, Inc. of Montvale, New Jersey. Work commenced during the month of November 1974 and was completed in the month of December 1974.

The actual work procedures followed closely the steps outlined in my November 8, 1974, report (pages 2 and 3) with only one (1) exception: Said exception being improper treatment of wide joint in Bay #9 between the wing wall and the concrete dam due to reputed adverse ground conditions. I was unaware of the omissions until it came to my attention after the job was completed.

The original fill placed along this joint was very pervious, predominantly of organic nature and extremely loose. The replacement of this undesirable material was not carried or could not be carried out as per recommendations. The selective impervious fill I sent to the job was dumped into the excavated trench under water and, in my judgment, was a wasted effort.

Continued . . . . . .

Attention: Mr. S. A. Wyle

The leak probably originated from this location, is still in evidence at the downstream end of the dam, and should not cause any alarm in the immediate future since it has been in existence for a number of years. However, I firmly suggest that remedial steps be taken during the spring of 1975. I recommend a relatively inexpensive and effective method of pressure volclay treatment along the wing wall. If this cannot stop the leakage, then pressure grouting should be considered.

My field investigation and observation on the dam on January 17, 1975, was very gratifying. The serious leaks in Bay #1 are completely stopped and the precariously weak concrete deck was repaired very effectively by constructing a concrete pier.

Following the completion of "crack" repairs underneath the deck of the dam, the seepages and slight leaks were still in evidence. To repair such cracks effectively under continual seepage, water pressure and with the lack of proper support during concrete setting are extremely difficult or, at its best, is of a temporary nature. However, all these slight leakages and seepages were arrested following the application of Bentonite on the upstream end of the dam.

Presently, the entire structure is in good rehabilitated condition and free of seepages and leakages, with the exception of Bay #9. Periodic inspections and minor repair should be carried out by your maintenance personnel and once a year thorough observations should be conducted by a competent engineer.

On the spillway, loose concrete was chipped out at the locations I indicated and new concrete was placed and anchored to the old and sound concrete surfaces by means of dowels. New concrete placements were toed-in against the old concrete spillway at the upstream face to bolster its stability.

Original spillway concrete exhibits large amounts of exposed gravel, lack of sand and cement contents, thus rendering itself to easy deterioration and scouring effect of the spillway water. However, the importance of maintaining the spillway concrete progressively in good condition is of a low priority item and repair work should only be conducted in case of major occurrences of deterioration or spalds as witnessed during the inspection before the work was undertaken.

Continued . . . . . .

Attention: Mr. S. A. Wyle

In conclusion, I am very much pleased to report to you that the steps taken to carry out the rehabilitation of Cranberry Dam proved to be effective and satisfactory. Therefore, it is my recommendation to adhere closely to such steps in conducting your future maintenance program related to the items of similar nature.

Very truly yours,

Gungor Bastug

Gungor M. Bastug, M.S., C.E.
Hackensack Water Company

GMB:mw

Enclosure (Drawing)

cc: Mr. G. M. Haskew, Jr.,
Hackensack Water Company
Mr. R. Gerber,
Hackensack Water Company

476 Lookout Avenue
Hackensack, New Jersey 07601
April 23, 1976

Pothat Water Company
Post Office Box 45
Sloatsburg, New York 10974

Attention: Mr. S. A. Wyle,
Vice President and
Secretary-Treasurer

Re: Potaque, Cranberry
Dams and Spillway

Dear Mr. Wyle:

I am submitting this report based upon the observations I have conducted at the site on April 21, 1976.

At the time of my visit, water levels in both reservoirs stood approximately six inches (6") below high water marks.

Cranberry Dam:

The rehabilitative work performed at the dam structure during the period November 1974 through December 1974 continued to serve very satisfactorily. Soffits of all sloping concrete deck, horitzontal supporting beams and buttresses in nine (9) bays exhibited sound structural integrity. Progressive deterioration of concrete prior to corrective work is under control and there is no visible change from my last report dated August 12, 1975.

The long presence of leakage in bay #9 is still in existence but I detected no change in the flow since my first observation on November 8, 1974. At the present time, my recommendation is to maintain periodic check and report when there is an appreciable increase in the flow and/or there is an evidence of silt conveyance.

Continued . . . . . .

Attention: Mr. S. A. Wyle

In bay #1 (northerly bay), I have noticed slight seepage along the protruding concrete located underneath the sloping deck. Even though the seepage is not serious in nature, the weight of protruding concrete from the deck is an added stress to the deck and should be eliminated. I am enclosing a sketch delineating the proposed work.

Lack of proper ground grading in some of the bays have contributed to the collection of trapped water. This condition can easily be alleviated with minimum amount of grading and hand-dug ditches to divert the trapped water to the sloping terrain located at the downstream.

Potaque Dam:

The earth embankment is in excellent condition and completely devoid of seepage and soft ground conditions. The serious flow at the toe of the embankment as reported to you following the recent heavy rainfall should not cause any concern as said flow of water is no more than a state of normal condition of any earth embankment subjected to an appreciable rainfall. In fact, the presence of yellowing grass and surface vegatation coupled with compacted soil condition is an excellent indication that the saturation line is well below the surface and carried down to the toe of the slope as it should.

Spillway:

Concrete repair work along the spillway is in very satisfactory condition. The accumulation of debris, branches and sawed-up logs at the downstream and growth of grass at the upstream ends should be removed to preserve the spillway flow capacity.

In conclusion, I am pleased to report to you that the observations I made on the three sites assured me of the continuance of proper functioning of your structures.

Continued . . . . . .

Attention: Mr. S. A. Wyle

Please do not hesitate to call on me if I can be of any further assistance to your diligent efforts in preserving and upgrading the safety and structural integrity of your impounding reservoirs.

Very truly yours,

Gungor Bastug

GB:wm

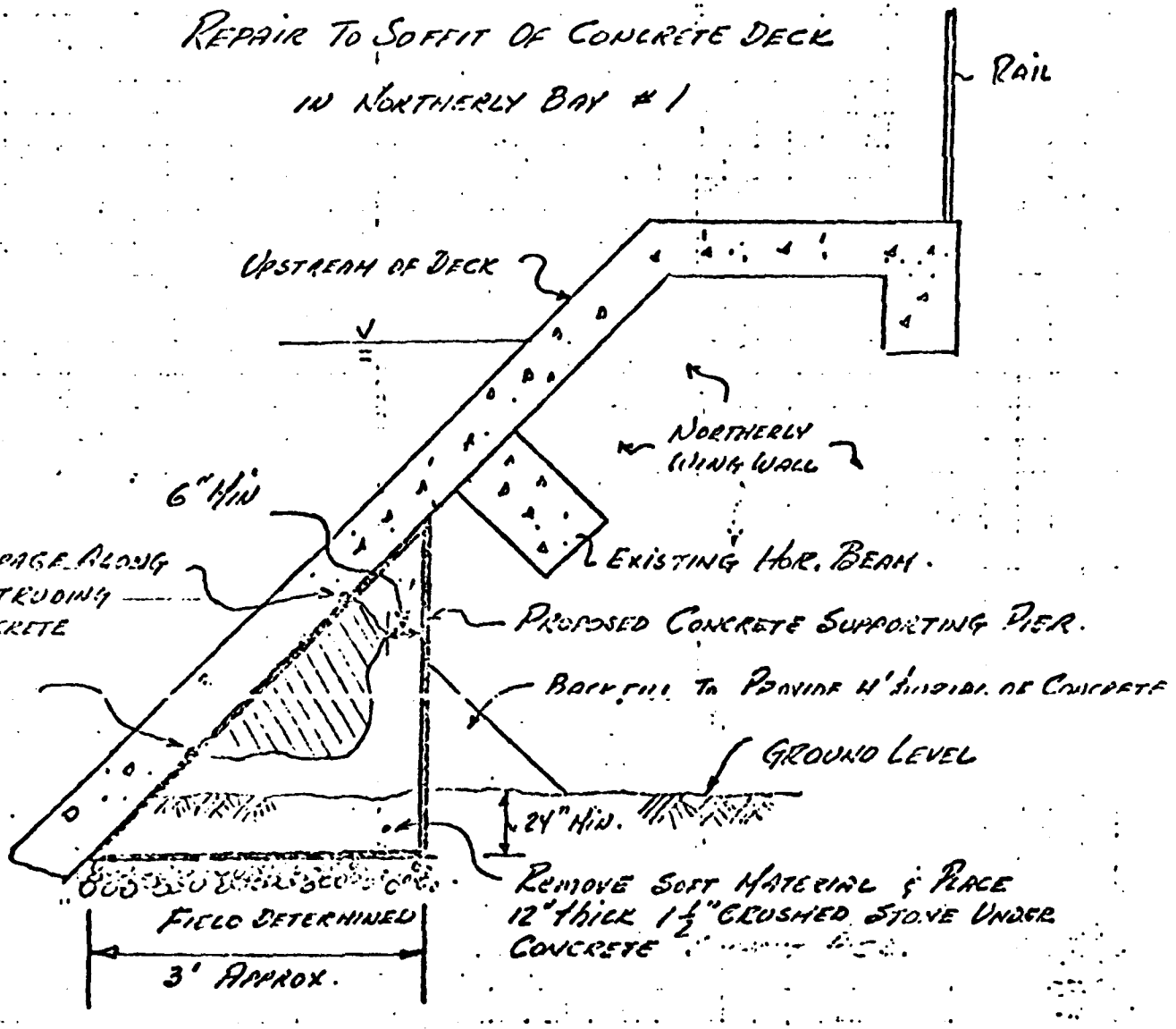
Enclosure

POTHAT WATER CO.

APRIL 22, 1976

CRANBERRY DAM

REPAIR TO SOFFIT OF CONCRETE DECK IN NORTHERLY BAY #1



CROSS SECTION - LOOKING NORTH

N.T.S.

NOTES:

1. LENGTH OF CONCRETE SUPPORTING PIER IS TO BE SUFFICIENT ENOUGH TO ENCASE THE PROTRUDING CONCRETE TOTALLY AND WITH MINIMUM CLEARANCE OF 6" FROM THE NEAREST POINT
2. REMOVE DETERIORATED WOOD FORMS ALONG THE UNDERSIDE PRIOR PLACING CONCRETE.

476 Lookout Avenue
Hackensack, New Jersey 07601
November 3, 1977

Pothat Water Company
Post Office Box 45
Sloatsburg, New York 10974

Attention: Mr. H. Pierson Mapes
President

Re: Rehabilitation of
Cranberry Dam and
Spillway

Gentlemen:

I am submitting this report to you based on my observation conducted on November 2, 1977.

Concrete dam structure continued to serve effectively since the completion of the rehabilitative work during December of 1974, and will continue to do so if periodic maintenance is implemented as suggested in my previous reports.

Presently, I recommend to perform the following items prior to inclement weather conditions:

1. Remove loose mortar layer above the water level on the upstream end of the dam and treat the areas with half inch thick layer of SIKADUR LO-MOD MORTAR.

2. "Vee" out cracks on the upstream face of the dam and seal them with SIKA COLMA-DUR LV.

3. The leak emanating within bay #9 (easterly end) is still in existence. Such leak can be attributed to a variety of reasons. If the possible reason is the separation between wing wall and the sloped deck of the dam, corrective measures should be taken in order to eliminate the most concerning cause, in my opinion. Therefore, temporary placement of common fill at the corner and excavating it to expose the joint is the first step to be taken. The second step is to treat the joint with COLMA Joint Sealer followed by impervious backfill and subsequent removal of the temporary fill to complete the work. In order to gain access to the site, four cedar trees must be removed.

Continued . . . . . .

Attention: Mr. H. Pierson Mapes

If a decision is made to follow up my recommendations as outlined in this report, I call your attention to a close supervision of the work.

Please feel free to contact me if I can be of any further assistance to you in the matter.

Very truly yours,

Gungor M. Bastug, M.S., C.E.

GMB/aj

476 Lookout Ave.
Hackensack, NJ 07601
September 7, 1978

Subject: Seepage and Piping at the Downstream End of Cranberry Dam

Pothat Water Company
P.O. Box 45
Sloatsburg, NY 10974

Att: Mr. H. Pierson Mapes, President

Gentlemen:

Based upon my recent site inspections, I would like to submit this report to your attention in order to bring certain facts to light, along with my related recommendations to further protect the structural integrity of the dam.

1. The leak which I observed in Bay #9 (Easterly End) during the latter part of 1974 is still in existence and has been in existence prior to my visit, per verbal statements from the people of your organization. As I have then stated, I did not put too much emphasis on this leakage, and I still do not consider it serious. The recent ditches which were excavated within the bay, pinpointed the leak under the footing of the buttress wall of the dam. Seepage carries no sedimentation and the flow rate showed no change. Since the joint between the wingwall and the dam buttress wall was not treated properly during the rehabilitative work conducted in November 1974, one of the reasons can be attributed to said improper treatment. The shoreline washout immediately at the wingwall location (one Birch tree was uprooted recently) certainly diminishes the path of resistance of the water infiltration around the wingwall, and may find its course to the bottom of the footing. The presence of the 6" water line, septic field and the natural ground water on the rising banks on the east side, can also be contributing factors. I recommend the following steps to be taken:

    1. Treat the joint between the buttress and wingwalls as outlined in my Nov. 8, 1974 report (Refer to item 5 on page 2).

    2. Backfill washout of the bank and place rip-rap to prevent further erosions.

    3. Take a sample of water and test for the presence of septic contamination and combined chlorine residual.

4. Consider pressure grouting under footing after analyzing steps 1, 2 and 3.

2. Another observation which requires further investigation is the "piping" (Hydrostatic boils) at the downstream end of the dam opposite to the joint buttress wall of bays #8 and 9. Two small "pipings" were uncovered in the ditches excavated recently to drain the puddled water from the bays. The existence of these boils can not be determined readily unless the following steps are taken as a first approach:

   1. Excavate at least a 2' deep trench in an "L" shape around them as explained to your personnel.

   2. Take sample and have them tested for septic contamination and combined chlorine residual.

If the results of these steps prove to be inconclusive, I shall make further recommendations to trace the source, which is more important than its existence. Boils exhibit very low hydrostatic pressure and the flow is minute, free of sediments, and can not be considered serious in nature at the present time.

3. Ingress and egress to the downstream end of the dam is by walking only. It is my further recommendation that an access road should be constructed from the west end to bring in equipment in order to perform some of the probable work I consider essential in the future. Such access road should be located at the proper alignment and under the supervision of a competent contractor. I shall render my suggestions and submit a proposed road alignment and profile drawing, if you wish to carry out my recommendation.

I have discussed the matter, in general, with C. Scott Vanderhoef of your organization and requested Kuchar to submit an estimated cost for treating the joint between the wing and buttress walls of Bay #9.

Do not hesitate to contact me if you have any questions concerning the matter.

Very truly yours,

Gungor Bastug

Gungor Bastug
Civil Engineer

GB:dl

476 Lookout Avenue
Hackensack, NJ 07601
October 21, 1978

Subject: Cranberry Dam

Pothat Water Company
P.O. Box 45
Sloatsburg, N.Y. 10974

Att: Mr. H. Pierson Mapes
President

Gentlemen:

This report and the enclosures will summarize the work conducted at the upstream and downstream end of the Cranberry Dam.

1. An access road starting from the north end of the dam (in the vicinity of abandoned tennis courts) and terminating along the bays of the downstream end of the dam, have been constructed to gain an access with the equipment. Said road followed the natural terrain, except for minimum cuts to avoid the sharp grades. The road has been stabilized by placing of 6"± slag material brought in from your property.

2. All 9 bays of the dam have been cleared of wild growth, concrete chips, spalds and other debris, and graded for proper drainage.

3. 4" perforated P.V.C. pipes with French drains have been installed to divert the water from some of the bays to the ditch at the outfall of the Sluice gate conduit.

4. The ditch at the outfall of the Sluice gate conduit was cleared of growth and debris. Proper slope was instituted for unobstructed flowage, and also to gain the necessary access to the Sluice gate for periodic inspection.

5. The washout on the south side of the upstream end has been repaired and rip-rapped with large to medium size slags.

6. Investigation of the continuous flow of water in Bay #8 was traced to the natural ground water table of the sharply sloped hill on the downstream end of the dam. Excavations within the bay confirmed the existence of glacial type of fill, predominantly granular material of varying sizes mixed with fine silty sand. The water table investigation holes excavated on the hill

beyond and within the limit of the dam structure, indicated the presence of the water table within 12" to 15" of the existing ground surface. This is a natural condition created by the water level of the impounded lake. Therefore the seepages, and water infiltrations observed in Bay #8 and along the toe of the sloping hills are due to the high ground water table and the type of the strata composition of the natural terrain.

These positive observations totally eliminate my previous concerning point of the probable piping under the dam structure.

The stability of the highly saturated hills should not be considered serious in view of its existence since the construction of the dam. Consequently, the matting of the ground vegetation and growth should never be disturbed as they are excellent protection against the erosion.

In the near future, I will investigate the condition of the 30" Sluice gate through the 30" ø concrete conduit.

Please do not hesitate to contact me if you have any questions on the subject matter.

Very truly yours,

Gungor Bastug

Gungor Bastug
Civil Engineer

GB/aj
Encls.

Tenth Ave Dam.

RECEIVED

APR 16 1980

SOILS SECTION

476 Lookout Avenue
Hackensack, N.J. 07601
April 4, 1979

Subject: Cranberry Dam

Pothat Water Company
P.O. Box 45
Sloatsburg, N.Y. 10974

Att: Mr. H. Pierson Mapes,
President

Gentlemen:

This is a follow-up report to my previous one dated October 21, 1978.

On April 4, 1979 investigation was made thru the 36"Ø reinforced concrete conduit located along the Bay #5 and the Sluice gate which was installed at the upstream end of the conduit at the time of the dam construction. The 36"Ø reinforced concrete conduit was found to be in structurally sound condition for its entire length of 47' 3" from the headwall to the Sluice gate location. There are a few spalls in the concrete, but not to a degree to reduce its safety. These spalls, at an oppurtune time, should be repaired to arrest further deterioration. The constant leakage at the outfall of the conduit is a direct result of ground water penetration thru the spalls.

The Sluice gate exhibited excellent condition without any leakage from the dam. The fabricated steel gate is firmly seated against the brass seat cast in the concrete conduit. Slight scale of rust at the gate is superficial and investigation proved that there was no pitting in the metal itself.

Various pictures were taken at the Sluice gate and inside the conduit to document the findings. Additional pictures were also taken at the downstream end of the dam for pictorial record. Said pictures will be forwarded to you under a seperate cover.

Further investigations were made at the site to verify the results of the work as outlined in my October 21, 1978 report. The access road is in very good condition. The French drains installed within and outside the bays of the dam continued to function very satisfactorily. All bays are dry and free of water puddles.

Pothat Water Company -2- April 4, 1979
Att: Mr. H. Pierson Mapes

Please enter a correction to my October 21, 1978 report in which 36"Ø conduit and Sluice gate was typed inadvertantly as 30" Ø.

If you have any questions concerning this report please do not hesitate to contact me.

Very truly yours,

Güngor Bastug,
Civil Engineer

GB/cp

Cranberry Dam and Spillway:

The hollow concrete dam is situated to the east of the 100 lake. It is approximately 135 ft. long and has a maximum heig of 25 ft. The overflow provision is handled by the existing 205 ft. long spillway structure.

This reinforced dam has been in existence over fifty years and is a "Ambursen" type which was widely accepted as one of most safe dams in the United States.

The upstream end of the dam is a 9" thick sloped, reinforced concrete deck supported by horizontal beams which were tied in the reinforced concrete buttresses. There are 9 bays at about 15 feet center to center and buttress foundations and dam toe-i cutoff wall were constructed on highly compacted glacial till.

My detailed investigation of the dam and periodic maintenanc work conducted over the last four years assures the safety of this structure for many decades to come.

Since the structure is open to visual inspections, I have no detected any failure of its members.

The watershed area of the lake is slightly larger than that of Potaque Lake and the spillway capacity exceeds 8,000 cubic feet per second with 16" freeboard at the dam. Therefore, the size of the spillway is more than ample and is well above the requirement of 100 year flood.

This report will not go into the detailed studes and the site investigations, and rehabilitative repairs conducted at the dam and the spillway to introduce additional upgrading of the structures. We have employed Professional Engineer Mr. Miles Kuchar, and his Company and benefitted from his ideas and quality workma ship. The same contractor is now conducting a large scale rehabilitative work on the concrete dam of Oradell Reservoir owned and operated by Hackensack Water Company of Bergen County.

As a Professional Engineer with a Master's degree in C.E. and twenty four (24) years experience in construction and investigative works, I have the responsibility to the public and to you as a consultant to report my professional judgement in a realistic manner. With periodic inspection and/or required minor work, both spillway and the dam will continue to serve satisfactorily and safely.

I am enclosing herewith U.S. Geological map, Sloatsburg quadrant delineating the subject lakes and dams mentioned in this report.

Please do not hesitate to contact me if you have any questions concerning the mattor.

Very truly yours,

Gungor Bastug M.S.C.E.

REFERENCES

APPENDIX G

References

1. "HEC-1 Flood Hydrograph Package for Dam Safety Investigations", U.S. Army Corps of Engineers, September 1978

2. "Lower Hudson River Basin Hydrolic Flood Routing Model" for New York District Corps of Engineers, Water Resources Engineers, Inc., January 1977

3. "Standard Project Flood Determination", EM-1110-2-1411, Army Corps of Engineers, Washington, D.C., Rev. 1965

4. "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian", Hydrometeorological Report No. 51, National Weather Service, June 1978

5. "National Program of Inspection of Dams", Vol. 3, Department of the Army, Office of the Chief of Engineers, 1975

6. "Flood Hydrograph Analyses and Computations", EM-1110-2-1405, U.S. Army Corps of Engineers, August, 1959

7. "Recommended Guidelines for Safety Inspection of Dams", Department of the Army, Office of the Chief of Engineers, Appendix D

8. "The Geology of New York State", by Broughton, J.E., et al., N.Y. State Museum and Science Service, Geological Survey, Albany, New York, Map and Chart Series: No. 5, 1962

9. "Soil Association Map of New York State", by M.G. Cline, New York State College of Agriculture, Cornell University, Ithaca, New York, February, 1963

10. "Orange County Soils. Soil Association Leaflet 2", by E.G. Knox, et al., New York State College of Agriculture, Cornell University, Ithaca, New York, October, 1954

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